PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXVII—No. 5

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May, 1911

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MAY, 1911

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1911.

American Society of Civil Engineers

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ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER.................5913 Columbus.
CABLE ADDRESS..... ...."Ceas, New York."

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

April 19th, 1911.—The meeting was called to order at 8.30 P. M.; A. L. Bowman, M. Am. Soc. C. E., in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 112 members and 13 guests.

A paper by James E. Howard, Esq., entitled "Some Tests of Large Steel Columns," was presented by the Acting Secretary, who also read communications on the subject by Messrs. Horace E. Horton, A. N. Talbot, James Christie, and N. R. McLure. The paper was discussed orally by Messrs. Albert Lucius, Leon S. Moisseiff, A. W. Carpenter, J. S. Branne, Lewis D. Rights, and J. R. Worcester.

The Acting Secretary announced the following deaths:

WILLIAM BION EWING, elected Junior, April 3d, 1889; Associate Member, February 3d, 1892; Member, November 2d, 1898; died April 8th, 1911.

Harvey Childs Lowrie, elected Member, March 2d, 1892; died March 26th, 1911.

Walter Edward Lydston, elected Junior, April 4th, 1905; died May 8th, 1910.

Adjourned.

May 3d, 1911.—The meeting was called to order at 8.30 P. M.; A. L. Bowman, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 152 members and 28 guests.

The minutes of the meetings of March 15th and April 5th, 1911, were approved as printed in *Proceedings* for April, 1911.

A paper entitled "Sinking a Wet Shaft," by John P. Hogan, Jun. Am. Soc. C. E., was presented by the author, who illustrated his remarks with lantern slides. Written communications on the subject by Messrs. Mason D. Pratt and Charles B. Buerger were read by the Secretary, and the paper was discussed orally by Messrs. L. White, H. M. Hale, H. F. Dunham, T. H. Wiggin, T. Kennard Thomson, Bertrand H. Wait, V. H. Hewes, and the author.

The Secretary announced the election of the following candidates on May 2d, 1911:

As Members.

Hermon Charles Allen, Great Falls, Mont.
Ralph Jesse Arey, Los Angeles, Cal.
Mortimer Elwyn Cooley, Ann Arbor, Mich.
William Rowe Farrington, Middleboro, Mass.
Ralph Seth Fessenden, Rupert, Idaho.
George Brodhead Harris, Wallingford, Pa.
George Beckley Hull, Ottawa, Ont., Canada.
Tetsuzo Kurashige, Tokyo, Japan.
Frank Lee, Winnipeg, Man., Canada.
George Mustin Lehman, Pittsburg, Pa.
William Baird Patton, Duluth, Minn.
Carl Edouard Pelz, New York City.
Louis Henry Rathman, Buffalo, N. Y.
Charles Lorraine Ruffin, Micaville, N. C.
Eugene Hicks Shipman, South Bethlehem, Pa.
Aubrey Weymouth, New York City.
Frederick Charles Williams, Cleveland, Ohio.

As Associate Members.

Carl Prescott Abbott, Valhalla, N. Y.
Arthur Edward Arledge, Honolulu, Hawaii.
Charles Lester Bailey, Fort Shaw, Mont.
George Ellsworth Barrows, Buffalo, N. Y.
Fred Thomson Bass, New York City.

Henry Rumrill Beebe, Utica, N. Y.
William Neville Collier, La Crosse, Wis.
Henry John Cox, New Iberia, La.
John Percival Davies, New York City.
Bernard Dall DeWeese, Denver, Colo.
Harry Dievendorf Dewell, Berkeley, Cal.
Gilbert Colfax Dobson, Gatun, Canal Zone, Panama.
Louis Garbi, Jr., Watertown, N. Y.
Nathan Jackson Gibbs, Porto Bello, Canal Zone, Panama.
Tresham Dames Gregg, Chicago, Ill.
Thomas Stephen Griffin, Little Hocking, Ohio.
William Francis Roelofson Griffith, Morristown, N. J.
Derwent Gordon Heslop, Shinchow-Fu, Kwangtung, South China.
Arthur Templeton Kenyon, Cristobal, Canal Zone, Panama.
Thomas Edward Lally, Boston, Mass.
Frank Oliver Price, Brooklyn, N. Y.
Jacob Brunn Reinhardt, Rochester, N. Y.
Eugene Eugenievich Skorniakoff, St. Petersburg, Russia.
Clifford Milton Stegner, Cincinnati, Ohio.
Adolf Stellhorn, Leavenworth, Kans.
Henry Wade Swanitz, San Francisco, Cal.
John Taylor, Ottawa, Ont., Canada.
Rufus Mason Whittet, Boston, Mass.
Robert Emmet Wise, Cold Spring, N. Y.
Otis Hord Wright, Portland, Ore.

As Associates.

William Alden Brown, Burham, England.
Charles Pearl Price, Boston, Mass.
William John Vanderkloot, Chicago, Ill.

As Juniors.

Otto William Julius Anschuetz, Leavenworth, Kans.
Paul Jones Bean, Portsmouth, Va.
William Patrick Feeley, Cleveland, Ohio.
Harold Hansen Fitting, Coyote, Cal.
Dwight Walton Leggett, Ashtabula, Ohio.
Edgar Henry Mix, Portland, Ore.
James Rex Pemberton, Washington, D. C.
Merle Bentley Piper, The Dalles, Ore.
Edgar Kingsbury Ruth, New York City.
Carl William Schedler, Jr., Mt. Vernon, N. Y.
Hymen Aaron Seltzer, St. Louis, Mo.

CURTIS PENDLETON SNOOK, Upper Montclair, N. J.
HUBERT EARL SNYDER, Gatun, Canal Zone, Panama.
ROY STANLEY SWINTON, Ann Arbor, Mich.
ROY ELSEN WARD, Massena, N. Y.
WILLIAM SATTERWHITE WHITMAN, Louisville, Ky.

The Secretary announced the transfer of the following candidates on May 2d, 1911:

FROM ASSOCIATE MEMBER TO MEMBER.

VARNUM PIERCE CURTIS, Worcester, Mass.
ISAAC HARBY, Trenton, N. J.
ADAM HUNTER, Glasgow, Scotland.
SETH MORTON VAN LOAN, Philadelphia, Pa.

FROM ASSOCIATE TO MEMBER.

JOHN VOSE HAZEN, Hanover, N. H.

FROM JUNIOR TO ASSOCIATE MEMBER.

FARRAND NORTHROP BENEDICT, New York City.
JULES ROWLEY BREUCHAUD, New York City.
GROVER CHARLES BROWN, Buffalo, N. Y.
EDGAR STONE CLOSSON, Brooklyn, N. Y.
CHARLES LOUIS FOX, Wilkinsburg, Pa.
NORMAN MARSHALL HALCOMBE, Stanford University, Cal.
JOHN BACON HUTCHINGS, JR., Urbana, Ill.
GRANDVILLE REYNARD JONES, Washington, D. C.
JOHN ENDICOTT PORTER, Yonkers, N. Y.
JAMES CHARLES FORSYTHE SHAFER, Hadlock, Wash.

The Secretary announced the following death:

RALPH BARTON MANTER, elected Associate Member, June 1st, 1904; died February 2d, 1911.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

May 2d, 1911.—President Endicott in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Belknap, Boller, Clarke, Kimball, Loomis, Loweth, Ridgway, Roberts, Snow, Talbot, and Thompson.

Preliminary action was taken looking to the participation of the Society in the 12th International Congress of Navigation, to be held at Philadelphia, Pa., in 1912, by official invitation of the United States Government.

Preliminary action was taken in regard to the appointment of a proposed Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities.

Action was taken in regard to members in arrears for dues.

Ballots for membership were canvassed, resulting in the election of 17 Members, 30 Associate Members, 3 Associates, and 16 Juniors, and the transfer of 10 Juniors to the grade of Associate Member.

Four Associate Members and 1 Associate were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

June 7th, 1911.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "Two Earth Dams of the United States Reclamation Service," by D. C. Henny, M. Am. Soc. C. E.; and "Steel Centering Used in the Construction of the Rocky River Bridge, Cleveland, Ohio," by Wilbur J. Watson, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for April, 1911.

September 6th, 1911.—8.30 P. M.—A regular business meeting will be held, and a paper by George B. Francis and Joseph H. O'Brien, Members, Am. Soc. C. E., entitled "The New York Tunnel Extension of the Pennsylvania Railroad: Certain Engineering Structures of the New York Terminal Area," will be presented for discussion.

This paper is printed in this number of *Proceedings.*

September 20th, 1911.—8.30 P. M.—A paper entitled "Mule-Back Reconnaissances," by William J. Millard, Jun. Am. Soc. C. E., will be presented for discussion at this meeting.

This paper is printed in this number of *Proceedings.*

October 18th, 1911.—8.30 P. M.—At this meeting a paper by George Gibbs, M. Am. Soc. C. E., entitled "The New York Tunnel Extension of the Pennsylvania Railroad: Station Construction, Road, Track, Yard Equipment, Electric Traction, and Locomotives," will be presented for discussion.

This paper is printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-third Annual Convention of the Society will be held at Chattanooga, Tenn., June 13th to 16th, 1911, inclusive.

A circular giving full information in reference to the Convention was issued on May 12th, 1911.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership in this matter is essential to the regular issue of each quarterly volume.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

SUBSCRIPTION PRICE TO THE PUBLICATIONS OF THE SOCIETY

The following subscription rates have been fixed by the Board of Direction for the publications of the Society:

Proceedings, ten Numbers per annum, $8. Price for single numbers, $1.

Transactions, four Volumes per annum, $12. Price for single volumes, $4.

On the above prices there is a discount of 50% to members who desire extra copies of any of these publications, and of 25% to Libraries and Book-dealers.

There is also an additional charge per annum, to cover foreign postage, of 75 cents for *Proceedings* and $1 for *Transactions,* or 8 cents and 25 cents, respectively, for single numbers.

A special subscription rate has been fixed by the Board for the *Proceedings* of the Society for the benefit of Students in Technical Schools. This rate is $4.50 per annum, and is available to any *bona fide* student of any technical school.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest of the Association at any of the above meetings, if he will notify the Secretary that he is in San Francisco.

(Abstract of Minutes of Meeting)

April 21st, 1911.—The meeting was called to order; President Derleth in the chair; E. T. Thurston, Jr., Secretary; and present, also, 63 members and guests.

The Secretary read an extract of a letter from Chas. Warren Hunt, Secretary of the Society, complimenting the Association on its success, and calling attention to the fact that of the net increase in membership of the Society of 544, from February 10th, 1910, to February 10th, 1911, 17% was from the States of California, Oregon and Wash-

ington, and that California has more members than any other State except New York and Pennsylvania. The Secretary also announced that the membership of the Association had reached 135.

On motion, duly seconded, the President was instructed to appoint a committee of three to confer with the other local engineering societies relative to the appointment of similar committees to form a general conference committee for the purpose of securing an International Engineering Congress in San Francisco in 1915. Subsequently, the President was appointed *ex-officio* Chairman of the proposed committee.

In accordance with instructions received at the February meeting, the President announced: that Messrs. B. P. Legare, C. Uhlig, W. A. Cattell, M. M. O'Shaughnessy, and J. H. Wallace, had been appointed a Committee to choose a site for the Panama-Pacific Exposition; that letters jointly signed by the Presidents of the four Engineering Societies had been addressed to the Directors of the Exposition; that the Association Committee had held two meetings at which the entire matter had been discussed; and that two meetings had been held with the Directors of the Exposition.

A paper giving an interesting and instructive description of the constructive and salient details of the Dumbarton Bridge of the Central California Railway of the Southern Pacific Company, was read by E. J. Schneider, M. Am. Soc. C. E., who illustrated his address with stereopticon views. The subject was discussed briefly by several members.

Adjourned.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Gavin N. Houston, M. Am. Soc. C. E., 409 Equitable Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, and until further notice, will take place at The Colorado Traffic Club.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

April 8th, 1911.—The meeting was called to order at 8 P. M.; Vice-President Comstock in the chair; H. J. Burt, Secretary; and present, also, 17 members and 6 guests.

A paper by F. C. Carstarphen, Assoc. M. Am. Soc. C. E., entitled "Some Notes on Sulphur," was presented by Gavin N. Houston, M. Am. Soc. C. E., after which W. F. Allison, Assoc. M. Am. Soc. C. E., presented a brief report of the sanitary works on the Isthmus of Panama, and a number of views of the Canal work were displayed.

Adjourned.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniørforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 302 Board of Trade Building, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 321 Hibernia Bank Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From April 9th to May 6th, 1911)

DONATIONS *

THE AMERICAN YEAR BOOK.

A Record of Events and Progress, 1910. Edited by S. N. D. North, Under Direction of a Supervisory Board Representing National Learned Societies. Cloth, 8 x 5½ in., 20 + 867 pp. New York and London, D. Appleton and Company, 1911. $3.50.

This publication, the preface states, is the first volume of what is expected to be an annual series, and is intended to be the work of a body of experts, each reviewing the field with which he is most familiar. It is said to be "a record of events and progress" and as such is intended to serve as a handy book of reference for students, writers, and searchers. The material is subdivided into departments by grouping the text into affiliated subjects. Cross-references are also freely used and a full and carefully analyzed index is provided. While the Year Book aims to record the progress of other countries, it is stated to be devoted chiefly to American progress. The Contents are: Comparative Statistics; History and Law; Government and Administration; Functions of Government; Economic and Social Questions; Industries and Occupations; Science and Engineering; The Humanities; Current Record; Index.

NOTES ON PLATE-GIRDER DESIGN.

By Clarence W. Hudson, M. Am. Soc. C. E. Cloth, 9¼ x 6 in., illus., 7 + 75 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1911. $1.50.

This book is said to contain the theoretical and practical information necessary to enable engineers and students in civil engineering to make a design and general detailed drawing for a through plate-girder railway bridge. The theory of the plate-girder has been developed, the author states, so that it may be applied to such structures for any duty. The Chapter headings are: Stress Distribution and General; Required Area of Cross-Section for the Flanges; The Design of the Cross-Section of the Flanges; Lengths of Cover Plates; Rivet Spacing in Girder Flanges; Web Plates; Area of Web Plates; Stiffeners for Web Plates; Splices for Web Plates; Splices for the Component Parts of the Flanges; Connecting One Girder to Another; End Bearings; Positions of Loading for Maximum Shear and Moment; Preparation of a Table of Bending Moments, Shears, and Concentrated Loads for Cooper's E_{50} Loading; Table of Moments, Shears, and Concentrated Loads for Cooper's E_{50}.

THE EARNING POWER OF RAILROADS, 1911.

Compiled and Edited by Floyd W. Mundy. Cloth, 7½ x 5 in., 492 pp. New York and Chicago, Jas. H. Oliphant & Co., 1911. $2.50.

The preface states that this volume contains important statistics and other facts relating to the earning powers and securities of 150 railroads (including practically all those in the United States, Canada, and Mexico), arranged in convenient form for ready reference. The Introductory Chapters are said to relate to the fundamental principles to be applied in investigating the value of the stocks or bonds of any railroad. Statistics regarding earnings, mileage, capitalization, tonnage, etc., are presented in Tables which are arranged to permit of easy comparison. These Tables are followed by Notes containing information relating to dividends, railroad capitalization, investments, physical and financial conditions, etc., of railroads which may be of interest to the investor. The official Annual Railroad Reports are stated to have been used almost exclusively in the preparation of this book. The Contents are: Income Account; Operating Expenses; Maintenance Expenses; Maintenance of Way; Maintenance of Equipment; Traffic, Transportation and General Expenses; The Operating Ratio; Fixed Charges; Stock Outstanding in its Relation to Earning Power; Guarantees and Their Relation to Surplus Available for Dividends; Tables; Notes.

EFFICIENCY AS A BASIS FOR OPERATION AND WAGES.

By Harrington Emerson. Cloth, 7½ x 5¼ in., 224 pp. New York, The Engineering Magazine, 1911. $2.00.

This development of works management, or the process of directing the great forces of manufacturing to the best advantage in economy, is stated to be a com-

* Unless otherwise specified, books in this list have been donated by the publishers.

plete statement of the elements of organization, management, and operation under the Efficiency or Individual Effort system, the methods advocated being practised in some of the largest manufacturing and operating institutions in the United States. These essays appeared originally as a series of articles in *The Engineering Magazine,* from July, 1908, to March, 1909, and are said to have been thoroughly revised, rewritten to a great extent, and much enlarged before publication in their present form. The Contents are: Typical Inefficiencies and Their Significance; National Efficiencies: Their Tendencies and Influence; The Strength and Weakness of Existing Systems of Organization; Line and Staff Organization in Industrial Concerns; Standards: Their Relations to Organization and to Results; The Realization of Standards in Practice; The Modern Theory of Cost Accounting; The Location and Elimination of Wastes; The Efficiency System in Operation; Standard Times and Bonus; What the Efficiency System May Accomplish; The Gospel of Efficiency.

RURAL HYGIENE.

By Henry N. Ogden, M. Am. Soc. C. E. Cloth, $7\frac{3}{4}$ x $5\frac{1}{4}$ in., illus., 17 + 434 pp. New York, The Macmillan Company, 1911. $1.50.

These pages are stated to deal with a systematic treatment of the question of individual living, together with those questions having to do with the cause and spread of disease, the transmission of bacteria from one community to another, etc. The structural side of public hygiene rather than the medical side is discussed, those chapters dealing with contagious diseases emphasizing the subjects of quarantine, disinfection and prevention rather than etiology and treatment. Descriptions of the various engineering methods are limited to suggestions as to proper selections of methods and right choice between processes, the details of construction being omitted. The Chapter headings are: Vital Statistics of Rural Life; Location of a House—Soil and Surroundings; Construction of Houses and Barns with Reference to Healthfulness; Ventilation; Quantity of Water Required for Domestic Use; Sources of Water-Supply; Quality of Water; Water-Works Construction; Plumbing; Sewage Disposal; Preparation and Care of Milk and Meat; Foods and Beverages; Personal Hygiene; Theories of Disease; Disinfection; Tuberculosis and Pneumonia; Typhoid Fever; Children's Diseases; Parasitical Diseases; Diseases Controlled by Antitoxins; Hygiene and Law; Index.

Gifts have also been received from the following:

Alabama-Board of Health. 2 pam.
Am. Telephone & Telegraph Co. 1 pam.
Arizona, Univ. of-Agri. Exper. Station. 1 pam.
L'Assoc. Amicale des Anciens Elèves de L'Ecole Centrale. 1 vol.
Babb, Cyrus C. 1 bound vol.
Brockton, Mass.-Water Commrs. 1 pam.
California-Board of Harbor Commrs. 1 pam.
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Royal Commission on Sewage Disposal: Seventh Report of the Commissioners Appointed to Inquire and Report What Methods of Treating and Disposing of Sewage (Including any Liquid from any Factory or Manufacturing Process) may Properly be Adopted. Vol. I. Published by His Majesty's Stationery Office, London, 1911.

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Civil Engineering at the Universal Exposition, Brussels, 1910. Permanent International Association of Navigation Congresses, Brussels.

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HAZEN, JOHN VOSE. Prof. of Civ. Eng. and Graphics, Dartmouth Coll., 33 North Main St., Hanover, N. H	Assoc. M.	June 5, 1889 May 2, 1911
HENDRICKS, VICTOR KING. Office Engr., Frisco Lines, St. Louis & San Francisco R. R. Co., Springfield, Mo	Assoc. M. M.	May 1, 1895 April 4, 1911
MACKINTOSH, LACHLAN. City Engr., Mandalay, Burma, India		Jan. 31, 1911
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ROBINSON, EDWARD WILLIAM. Vice-Pres., Ruggles-Robinson Co., 331 Madison Ave., New York City.

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SMITH, ELIOT NICHOLS. Stone Ridge, Ulster Co., N. Y.

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WOLCOTT, CHRISTOPHER STANTON. Care, Y. M. C. A., Hazleton, Pa.

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JUNIORS

BACKUS, MURRAY JAMES. Guanica Centrale, Ensenada, Porto Rico.

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BIGGS, CARROLL ADDISON. Designer for William G. Fargo, 219 West Franklin St., Jackson, Mich.

JUNIORS (*Continued*)

BROWN, GROVER CHARLES. Care, Enrique Ruiz Williams, Luz Caballero 16, Apartado 149, Sagua La Grande, Cuba.

BROWN, WILLIAM CLINTON. Care, Humphrey Gas Pump Co., S. A. & K. Bldg., Syracuse, N. Y.

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MACK, GEORGE HORACE. Field Draftsman, Davenport-Muscatine Ry., Care, K. C. Weedin, Davenport, Iowa.

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WESTON, FREDERICK SAMPSON. Middleboro, Mass.

WRIGHT, THOMAS JUDSON, JR. Office of Chf. Engr., The Piedmont Traction Co., Charlotte, N. C.

RESIGNATIONS

MEMBERS

	Date of Resignation.
KENLY, WILLIAM LACY	May 2, 1911

DEATHS

EWING, WILLIAM BION. Elected Junior, April 3d, 1889; Associate Member, February 3d, 1892; Member, November 2d, 1898; died April 8th, 1911.

LOWRIE, HARVEY CHILDS. Elected Member, March 2d, 1892; died March 26th, 1911.

MANTER, RALPH BARTON. Elected Associate Member, June 1st, 1904; died February 2d, 1911.

Total Membership of the Society, May 9th, 1911, 6 000.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(April 8th to May 5th, 1911)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spèciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $4.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian,* London, England.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions,* Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 20c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 15c.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England.
(70) *Engineering Review,* New York City, 10c.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 10c.
(77) *Journal,* Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen,* Vienna, Austria.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(83) *Progressive Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Roadmaster and Foreman,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 15c.
(97) *Turbine,* Berlin, Germany, 1 Mark.
(98) *Journal,* Engrs. Soc. Pa., 219 Market St., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $1.50.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., $1.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.

LIST OF ARTICLES.

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Bascule Bridge at Copenhagen Harbour.* (11) Mar. 31.
Relation of Bridge Specifications to Highway Improvement.* Albert Smith. (Paper read before the Indiana Eng. Soc.) (60) Apr.
Reinforcement of Pecos Viaduct. W. H. Alderson. (4) Apr.
Lift-Span of the Hawthorne Avenue Bridge, Portland, Ore.* (14) Apr. 8.
The Old Man River Viaduct.* (14) Apr. 8.
Concrete Bridges and Culverts in Iowa. T. H. McDonald. (Abstract of paper read before the Iowa Asoc. of Cement Users.) (14) Apr. 8.
A Suspension Bridge with Flat Cables of Riveted-Plate Construction. (13) Apr. 13.
Adaptation of Concrete to Long Span Bridges. F. Barber, A. M. Can. Soc. C. E. (Paper read before the Can. Cement and Concrete Assoc.) (96) Apr. 13; (86) Apr. 26; (14) Apr. 8.
A Reinforced-Concrete Arch Bridge with Separately Molded Members.* (13) Apr. 13.
Main Street Steel Arch Viaduct over O. K. Creek Valley, Kansas City, Mo.* Kenneth Hartley. (13) Apr. 13.

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Bridges—(Continued).

Renewal of the Gray's Ferry Bridge, Philadelphia.* S. Savage. **(14)** Apr. 15.
A Concrete Truss Bridge.* **(14)** Apr. 15.
Building Bridge Piers with Little Clearance. **(14)** Apr. 15.
Designs for the New Quebec Bridge, and the Accepted Design.* **(13)** Apr. 20.
Waterford Bridge.* **(11)** Apr. 21.
Anchor Pier Bearings and Typical Connections, Beaver Bridge.* **(14)** Apr. 22.
Recent Developments in Bridge Construction. Frank P. McKibben. (Paper read before Mass. Inst. of Tech.) **(86)** Apr. 26.
Impact Tests on a Reinforced Concrete Trestle, C., M. & St. P. Ry.* J. H. Prior. **(13)** Apr. 27.
Renewal of Kentucky River High Bridge.* H. H. Starr, Assoc. M. Am. Soc. C. E. **(13)** Apr. 27; **(15)** Apr. 28; **(11)** Apr. 29.
Reinforced Concrete Trestle on the Rock Island Railway.* **(14)** Apr. 29.
Substructure of the New Kentucky & Indiana Bridge.* **(14)** Apr. 29.
The Centering for the 281 Ft. Concrete Arch of the Monroe St. Bridge, Spokane, Wash.* P. F. Kennedy. **(13)** May 4; **(14)** Apr. 29; **(86)** Apr. 19.
Bridge Replacement on the Boston & Albany.* **(15)** May 5.
Dispositif de Manœuvre d'un Pont Tournant par Pivot à Vis.* A. Carrez. **(30)** Apr.
Le Double Pont Tournant dit *Herrenbrücke* sur la Trave à Lübeck (Allemagne).* L. Descans. **(30)** Apr.
Neubau der Langenzugbrücke in Hamburg.* Leo. **(78)** Apr. 1.
Vereinfachte Berechnung von eingespannten Gewölben nach der Elastizitätstheorie. S. Sor. **(78)** Apr. 1.
Der Umbau der Eisenbahnbrücke über die Elbe bei Wittenberge.* R. Wüstehube. **(48)** Apr. 15.
Einige neuere eiserne Brücken in Russland, insbesondere die Dnjepr-Brücke bei Alexandrowsk.* Karl Bernhard. **(51)** Apr. 19.

Electrical.

Electricity Applied to a Modern Tunnelling Work. Harry Edward Yerbury, M. Inst. C. E. **(63)** Vol. 183.
The Hygroscopic Susceptibility of Fibrous Insulating Materials for Electrical Machinery. William Pollard Digby. **(63)** Vol. 183.
Aerial Telegraphs on English Railways.* **(26)** Mar. 31.
Small Electric Lighting Plants. C. Bell Walker. (Abstract of paper read before the Birmingham and District Electric Club.) **(73)** Mar. 31.
On Resistances With Current and Potential Terminals.* G. F. C. Searle. **(73)** Serial beginning Mar. 31.
The Application of Electricity in Mining Industries.* Rollin W. Hutchinson, Jr. **(9)** Apr.
The Rise in Temperature of Electrical Apparatus.* F. Bacon. **(26)** Apr. 7.
Characteristics of the Three-Phase Series Commutator Motor.* R. Rudenberg. (Abstract from *Elektrotechnische Zeitschrift.*) **(73)** Apr. 7.
The Reduced Current Method for Localising Fractures in Submarine Cables.* R. Rolland Black. **(73)** Apr. 7.
A Mechanical Alternating-Current Wave-Analyzer.* C. A. Pierce. **(27)** Apr. 13.
Pressure Rises on Opening Short-Circuits.* A. G. Collis. (Paper read before the South Wales Inst. of Engrs.) **(27)** Apr. 13.
Specifications for New National Electrical Code Rubber-Covered Wire. **(27)** Apr. 13; **(13)** Apr. 27.
Submersible Electric Motor.* **(96)** Apr. 13.
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The Electrical Production of Steel at the Works of the Eicher-Hüttenvereins Le Gallais Metz & Co., at Dommeldingen. (Abstract from *Electrotechnische Zeitschrift.*) **(73)** Apr. 14.
Automatic Starters for Induction Motors.* R. H. Fenkhausen. **(64)** Apr. 18.
Methods of Guying and Anchoring Telephone Poles.* (From *W. & M. Telephone Wire News.*) **(86)** Apr. 19.
Simple Method of Making Thermo-Electric Measurements of High Temperature.* J. P. Raymond. **(27)** Apr. 20.
Losses in Single-Phase Rotors.* H. Weichsel. **(27)** Serial beginning Apr. 20.
Underground Telephone and Telegraph Cable Between Philadelphia and Washington.* **(27)** Apr. 20.
Diminutive Central Station at Lewis, Ia., Gasoline-Electric Equipment Supplying Energy to a Town of 650 Inhabitants.* J. H. Kuhns. **(27)** Apr. 20.
The Scherbius System of Speed Regulation for Induction Motors.* J. J. Elink Schnurman. **(27)** Apr. 20.
Electric Winding Plant at South Kenmuir Colliery.* **(22)** Apr. 21.
Electrically-Driven Reversing Mills at the Works of Messrs. Alfred Hickman.* **(26)** Apr. 21; **(73)** Apr. 14.

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Present-Day Methods of Protecting High-Tension Systems by Means of Automatic Switchgear.* (73) Apr. 21.
High-Tension Cables.* Paul Humann. (Abstract from *Elektrotechnische Zeitschrift.*) (73) Apr. 21.
Notes on D. C. Three-Wire Faults and Their Location. Walter E. Rogers. (26) Apr. 21.
Wireless Telegraphy and Airships.* (19) Apr. 22.
Hydro-Electric Development in Georgia—Plant of the Central Georgia Power Company at Lloyd Shoals, Ga.* (27) Apr. 27.
Excessive Rates Due to Holding Company. (27) Apr. 27.
Electric Welding and the Fusing of Metals.* A. S. Hatch. (27) Apr. 27.
Field Construction of a 110 000-Volt Transmission Line.* (13) Apr. 27.
Ornamental Curb Lighting at Hamilton, Ontario.* (27) Apr. 27.
School and Library Lighting. (27) Apr. 27.
Lighting and Electrical Equipment of a Central-Station Office Building.* Ross B. Mateer. (27) Apr. 27.
The Application of Heyland's Vector Diagram for Testing Induction Motors.* J. W. Rogers (10) May.
Public Gain from Improved Efficiency of Electric Lighting. William H. Blood, Jr. (Paper read before the Mass. Inst. of Tech.) (60) May.
Excessive Rates for Gas and Electric Lighting through the Operation of a Holding Corporation. (13) May 4.
Electrical Equipment of a Modern English Theater.* J. A. Seager. (27) May 4.
Sur la Conductivité Electrique des Alliages.* Witold Broniewski. (93) Apr.
Les Propriétés Diélectriques de l'Huile.* (33) Apr. 22.
Zur Kenntnis der Funkenspannung bei technischem Wechselstrom.* W. Weicker. (48) Apr. 8.
Lichtmaste in Eisenbeton.* Karl Rössle. (78) Apr. 20.

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Taikoo Dockyard, Hong Kong.* Albert Edwin Griffin, Assoc. M. Inst. C. E. (63) Vol. 183.
A 22 000-Ton Floating Dry Dock for Brazil.* (95) Jan.
Naval Engineering Progress. H. I. Cone. (Abstract of lecture delivered at Naval War College at Newport, R. I.) (95) Jan.
A Tank Steamer for the Molasses Trade.* (95) Jan.
Test of a Mosher Marine Boiler. (95) Feb.
The Economical Working of Reciprocating Marine Engines and Their Auxiliaries.* D. B. Morrison. (Paper read before the North-East Coast Inst. of Engrs. and Shipbuilders.) (95) Serial beginning Feb.
Modern Methods of Coaling Vessels.* (95) Feb.
New British Battleship *Conqueror*.* (95) Feb.
Torpedo Boat Destroyers *Perkins* and *Serrett*.* (95) Feb.
New Dry-Dock of the Toledo Shipbuilding Company.* (95) Feb.
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Random Notes on a Lake Freighter.* Casper F. Goodrich. (95) Apr.
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A Flying Motor Boat.* Frank C. Perkins. (95) Apr.
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Passenger and Freight Steamship *Suwanee*.* (95) Apr.
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Remarkable Economy of an Oil Fuel Installation.* (95) Apr.
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*Illustrated.

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French Auxiliary Motor Ships.* J. Peltier. (95) May.
Clay-Cutting Hydraulic Dredger for the River Nile.* (95) May.
Models of Vessels.* Charles B. Brewer. (10) May.
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The Pitot Tube as a Marine Speedometer and its Development.* Frank B. Sanborn, M. Am. Soc. C. E. (13) May 4.

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Practical Considerations in Gear Design and Construction.* Ralph E. Flanders. (8) Apr.
Universal Portland Cement Mills. (67) Apr.
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An Engineering Solution of Freight-Handling Problems.* H. McL. Harding. (9) Apr.
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Wheels, Ancient and Modern, and Their Manufacture.* Henry L. Heathcote. (29) Apr. 7.
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The Mechanical Design of Induction Motors.* H. L. Smith, A. M. Inst. C. E. (Paper read before the Rugby Eng. Soc.) (47) Serial beginning Apr. 7.
Some Automatic Recording and Indicating Instruments.* (47) Apr. 7.
Gas Producers.* W. A. Tookey. (Abstract of paper read before the Assoc. of Engrs.-in-Charge.) (47) Serial beginning Apr. 7.
The Making and Casting of Aluminum-Bronze. (From *The Brass-World.*) (47) Apr. 7.
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A Machine for Applying the Finishing Coat on Concrete Curb and Gutter.* (86) Apr. 12.
An Air Blast Apparatus for Depositing Concrete and Mortar Mixtures.* (86) Apr. 12.
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The Hauriot Monoplane, The Construction of a Remarkable French Flying Machine.* (19) Apr. 15.
Smoke Abatement by Supervision of Plant Construction. (14) Apr. 15.
Power Plant for Clay Working Industries. Ellis Lovejoy. (76) Apr. 15.
The Operation of the Ammonia Still. A. F. Blossey. (Paper read before the Ill. Gas Assoc.) (83) Apr. 15.
Syracuse Generating Station. Ralph E. Hecker. (Paper read before the Syracuse Section N. C. G. A.) (83) Apr. 15.
The Utility of the Pyrometer on Carburetted Water Gas Machines.* Chester S. Heath. (Paper read before the Ill. Gas Assoc.) (83) Apr. 15.
Modern Types of Elevators. F. E. Town. (Abstract of paper read before Pittsburgh Mfrs. and Contrs. Club.) (62) Apr. 17.
The Steam Turbine in Germany.* F. E. Junge and E. Heinrich. (64) Apr. 18.
Profitable Refinements in Contractors' Compressed-Air Plants.* Frank Richards, M. Am. Soc. M. E. (13) Apr. 20.
Reducing Motions for Steam Engine Indicators.* James A. Seager, Assoc. M. Inst. C. E. (13) Apr. 20.
The Algoma Steel Company's Coke Plant.* D. M. Griffith. (20) Apr. 20.
The Desiccation of Blast Furnace Air by the Use of Exhaust Steam.* (22) Apr. 21.

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Advantages of Long-Stroke Gasoline Engines for Commercial Vehicles. Edward A. Myers. (Abstract of paper read before the Soc. of Automobile Engrs.) (13) May 4.
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Turbine Pumps for Collieries.* R. H. Willis. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Apr. 21.
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Modern Ore Dressing Practice in Gilpin County, Colorado.* H. C. Parmelee. (105) May.
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Office Building and Carhouse of the Denver City Tramway Company.* John Evans. (17) Apr. 15.
The Transportation Problem of Greater Cleveland. A. B. Du Pont. (Paper read before Cleveland Eng. Soc.) (96) Apr. 20.
The "Dolter" Surface Contact System of Tramways in Torquay. Henry A. Garrett, Assoc. M. Inst. C. E. (Paper read before Inst. of Municipal and County Engrs.) (96) Apr. 20.
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Sewer Construction at Regina, Saskatchewan, Canada.* W. R. Harris. (Paper read before the Am. Soc. of Eng. Contractors.) (96) Apr. 13.
Difficult Reconstruction of a Large Sewer in Washington.* Asa E. Phillips. (14) Apr. 15.
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An Epidemic of Typhoid Fever at Lexington, Va., Evidently caused by Leaks from Sewer to Water Mains.* A. W. Freeman and Richard Messer. (13) Apr. 27.
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Solid Ceilings Between Iron Girders.* Robert Grimshaw. (19) May 6.
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Vol. XXXVII. JANUARY, 1911. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXVII. JANUARY, 1911. No. 1.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE PITTSBURG AND LAKE ERIE RAILROAD CANTILEVER BRIDGE OVER THE OHIO RIVER AT BEAVER, PA.

By Albert R. Raymer, M. Am. Soc. C. E.

To be Presented March 1st, 1911.

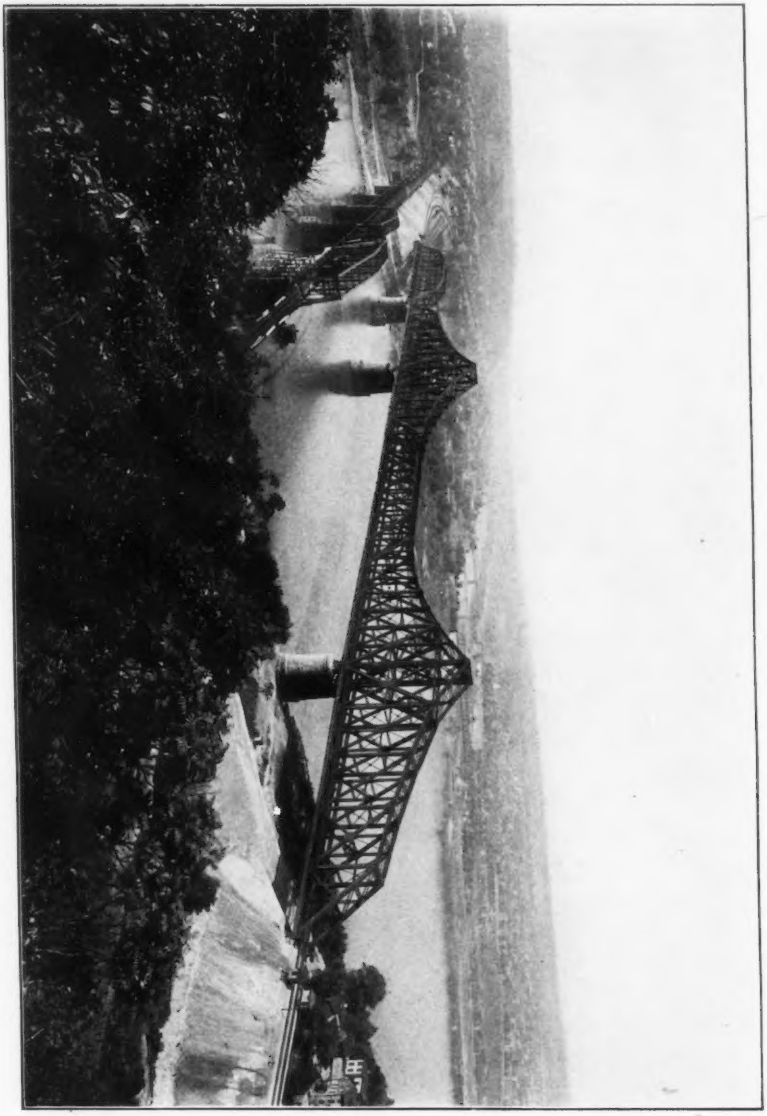
History and General Information.

As the business of a railroad increases it becomes necessary in many cases to change its facilities, and frequently a change in one department necessitates many and serious changes in others. The Pittsburg and Lake Erie—the road of the New York Central Lines into Pittsburg—is a striking example, for its management has spent vast sums of money to keep it abreast or a little ahead of the requirements of increasing traffic. Its bridge over the Ohio River at Beaver, Pa., built in 1878, had to be renewed by 1890, which was done on the single-track masonry by building a bridge which at that time was thought to have strength sufficient for many years. It was built for loadings equal to Cooper's E-30, with low unit stresses.

About 1897 the locomotives were increased in weight enough to allow the train tonnage to be increased from 1 600 to 2 500, and about 1902 the locomotives were again increased, with a corresponding train tonnage increase to 3 500, and since that date it has been increased to

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

PLATE I.
PAPERS, AM. SOC. C. E.
JANUARY, 1911.
RAYMER ON
RAILROAD CANTILEVER BRIDGE.



THE PITTSBURG AND LAKE ERIE RAILROAD CANTILEVER BRIDGE OVER THE OHIO RIVER AT BEAVER, PA.

more than 4 200. With this increase in tonnage, the tracks have been changed: from a single-track to a double-track approach to the bridge, with a single track on the bridge; then to a single-track gauntlet; and then to a four-track construction to points near the bridge. These conditions made it desirable to consider the reconstruction of the bridge, for strength as well as for increased capacity. The existing masonry piers were not large enough to support a double-track bridge, and, as they were constructed on timber grillage resting on gravel, it would not have been an easy matter to enlarge the piers under traffic; and, in addition, the River Interests desired the removal of one of the channel piers which was considerably in the way when large coal fleets were being handled at that point. After a careful study of the situation, a new bridge on a new location was recommended and approved in 1906. The new location is about 300 ft. up stream from the old bridge (Plate I), and the approaches are on a 3° curve, instead of 6°, as in the old approaches.

The Ohio River Bridge Laws require 90 ft. in the clear above low water, in the district in which this bridge is located. The bridge is on a summit in the grades, with a maximum grade approach of 0.30% on one side for about 2 miles.

The Government work of improving the river above and below the bridge site is planned for a channel through the bear-trap dams 700 ft. wide. This width is provided in the clear at the new bridge at right angles to the channel. After allowing for the skew and the thickness of the channel piers, the channel span was defined at 769 ft. from center to center of bearings. The channel span was located by the United States Government Engineers—after several meetings had been held to determine the wishes of the River Interests—with one pier just back of a timber crib on the south or left shore of the river. The length of the channel span, and the stipulation by the Government that no false-work should be placed in the channel in the river, practically determined the design to be a cantilever, and the location of the channel span forced one arm entirely over land, making it necessary, much to the regret of the Railroad Engineers, to use an approach span at the north or Beaver end. It was shown by the Railroad Engineers that floats, liberated at the coal-boat stage of water in the center of the channel a mile above the proposed structure would strike the channel pier, and yet the rivermen preferred the old channel, close to shore, defined

largely by the location of the offending pier in the old structure which was to be removed.

The spans and the general outline of the proposed bridge were determined by the Railroad Company's Chief Engineer, and the plans and general drawings were developed by Albert Lucius, M. Am. Soc. C. E., Consulting Bridge Engineer for the Railroad Company.

General Features of Design.

The general design of the new bridge called for two gauntlets of two tracks each to accommodate the four tracks at each end of the bridge, the northward passenger and the northward freight forming one gauntlet, and the two southward tracks making the other. The tracks in each gauntlet have their adjacent rails 1 ft. apart from center to center, both freight tracks being toward the center of the bridge and 13 ft. apart, the passenger tracks, therefore, are 15 ft. apart. The side clearances are 7½ ft. from the centers of the passenger tracks to the trusses, and 7 ft. to the skid girders; the overhead clearance is 21 ft. 6 in. from the top of rail.

The general loading assumed is Cooper's E-60 for the floor, and the same with a 10% reduction for the trusses.

As the E-60 loading, with full allowances for impact, wind, etc., is considered sufficient to provide for all estimated increases in train loads, rather high unit stresses were used.

River clearances and grade conditions made it desirable to have a shallow floor. The depth finally determined was 6 ft. from the clearance line to the top of rail. This depth practically determined the panel lengths for good stringer and floor-beam construction. The anchor arms were divided into ten panels of 32 ft. each; the cantilever arms into four panels of 32 ft. and four of 28 ft. 6 in. each; and the suspended span into ten panels of 28 ft. 6 in. each. This made the length of the anchor arms 320 ft., the cantilever arms 252 ft., and the suspended span 285 ft. The approach span is 370 ft. long, and is divided into twelve panels, each 30 ft. 10 in. long. The trusses are 34½ ft. apart from center to center, and 30 ft. apart in the clear. The floor is made up of nine lines of stringers, four under each gauntlet track and one half way between. A special feature of the floor is its strength, which is designed to take care of derailments under all probable conditions. Skid girders—the design of the Chief Engineer

FIG. 1.—MAIN TOWER POSTS, BRIDGE AT BEAVER, PA.

FIG. 2.—PART OF MAIN TOWER POST.

of the Railroad Company—have been used to prevent derailed equipment from striking the trusses, the object being to cause the car bodies to "skid" on the floor or against the skid girders until the momentum has been destroyed. These girders are double on each truss, and are placed at the proper height to give the best protection to the truss members.

The magnitude of the bridge, and the heavy loading, developed enormous stresses, which made it necessary to give much more than the usual attention to many details of the design, which will be considered at length at the proper place. Some of these features were developed in designing the anchorage, in transferring the wind stresses to the anchor masonry, and in the construction of the main-post bearings on the masonry. At the anchor points the anchor eye-bars are maintained under maximum stress at all times. The change in the position of the L_0 point over the anchorage, due to expansion and contraction, is prevented from causing bending moments in the eye-bars; the change in stress at the L_0 point from positive to negative does not cause any pounding, and the wind stress does not cause any stress at the anchor masonry, sidewise or otherwise, in the anchor eye-bars, when the abutment reaction is zero. These points have been protected by very ingenious details.

The main-post supports on the channel piers are unique in their design, in distributing the loads of 6 000 tons at each bearing definitely on the masonry under all conditions, and at the same time they prevent the development of secondary stresses of any serious magnitude in the lower chords at these points, due to the changes in the live loads in either magnitude or position.

The timber deck design has been made to serve, not only under normal conditions, but also to offer a large degree of safety under derailed equipment; the ties are well supported under the probable locations of derailed wheels, and they are prevented from bunching under such conditions. The timber in the deck has been treated chemically.

The anchor arm being comparatively long, the reaction at its end from dead load is very small, causing an uplift of only 33 000 lb. The uplift per truss at the L_0 point, caused by the live load and impact, amounts to 2 249 000 lb., and the load at the same point is 2 071 000 lb. This causes a reversal of stress in the web members up

to and including L_2, U_4, in the top chords up to U_6, and in the bottom chords up to L_4. The connections, L_0, L_2, U_2, and U_4, therefore, are riveted throughout. In all other cases the post and hanger connections to the bottom chords are riveted, while the main diagonals are connected to the bottom chords by pins back of the floor-beam connections, shown on the typical details of the panel points, L_{14} and L_{16}, Plate II. This was done in order to get an efficient connection of the floor-beams to the trusses, which otherwise, on account of the small available depth of 6 ft. from the top of rail to the clearance line, would have been very difficult, if at all possible. The members of the secondary system were also riveted to the bottom chords. In order to make the trusses of the anchor arm self-supporting during erection, and before the cantilever arm was erected, U_6-U_8, was also designed as a stiff member. It was constructed as a riveted four-web chord with pin connections at both ends, all other top chords being two-web chords. This provision was a source of great comfort when high water and driftwood threatened the falsework on the Beaver side after the anchor arm was swung, and before the cantilever was built, it having been just started.

The riveted connections in tension and in compression are fully spliced; no allowance was made for butt joints, with the exception of the member L_{10}-U_{10}, where, in addition to the straight butt joint, only half of the stress was taken care of by splice material and rivets. This member being vertical, the individual pieces forming it naturally had to come to a full bearing by their own weight.

The bottom chords have an **H** section, made up of twelve angles and two plates throughout. The heaviest bottom chord has 644½ sq. in., and is made up of three 60 by ⅞-in. web plates on each side, outer angles 8 by 8 by ¾-in., and inner angles 8 by 8 by ⅞-in., at top and bottom of the chord, and two lines of 8 by 8 by ⅞-in. angles on the inside of the chord.

The bottom chords have three sets of 6½ by $\frac{9}{16}$-in. double lacing, at the top and bottom, and a double lacing of 7 by 3½ by $\frac{7}{16}$-in. angles between the angles on the center line of the chord. Wherever the lacing connects the inner angles of the top and bottom flanges, tie-plates are used to connect with the outer angles. Vertical diaphragms, at from 5 to 6-ft. centers, are used in these chords throughout. The intermediate diaphragms are made in two separate parts in a vertical plane to clear the middle lacing. At the panel points the diaphragms

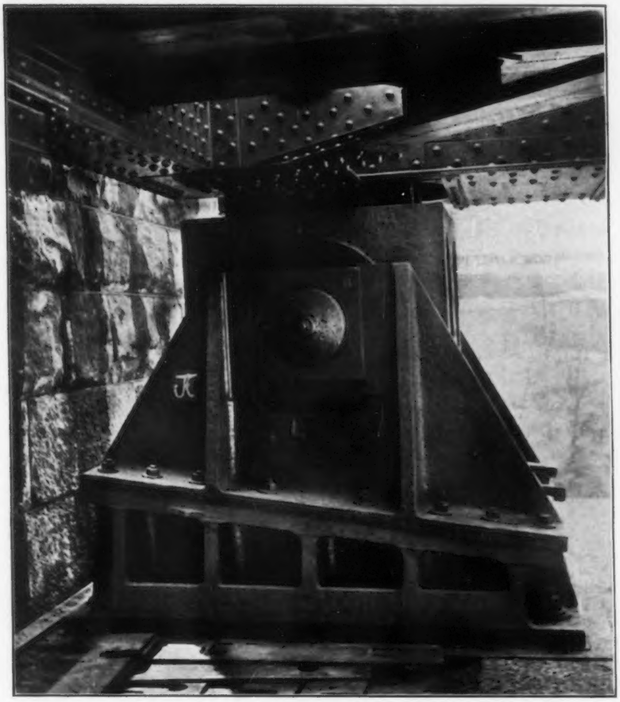


FIG. 1.—ANCHORAGE DETAILS, BRIDGE AT BEAVER, PA.

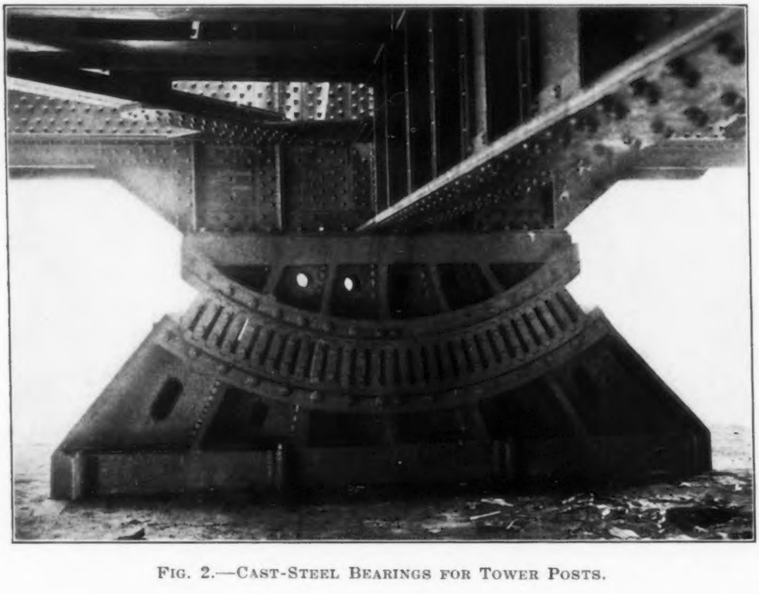


FIG. 2.—CAST-STEEL BEARINGS FOR TOWER POSTS.

are framed around the chord angles at the center of the chord, and riveted together on the tie-plate at the panel point. The chords are 3 ft. 6½ in. from back to back of the outer flange angles throughout.

The splices of the lower chord were detailed so as to permit of milling together all joints, for the purpose of maintaining exact lengths by the introduction of interior follower or splice-plates, which were not shop-riveted to either section of the splice. These facilitated the proper assembling of all sections in the shop and in the field, and yet distributed the splice values of section and of shear over more than one point, and put rivets in double shear. The connections of the web and the chord members were through large plates treated in a similar way. All connection plates were proportioned, not only for section and rivet shear in axial direction, but for the necessary section along shear lines between the several members assembled to a main joint, and also for the symmetrical absorption of stress in relation to their axes and to the resultant, through rivet values. The secondary bending and shearing stresses developing in large attachment plates, due to deviations of centers of gravity, were considered and guarded against by additional reinforcements.

The general design of the bridge provides for trusses riveted for full section of all joints and splices of chords, posts, and individual members, excepting as noted for the main-tower posts. Connections of chord and web members exposed to reversal of stresses are riveted joints, and so are all sub-panel connections and the end posts of the cantilever arms. Connections of chord and web members having stress in one direction only are pin-connected. In all the heavy members the lacing is on the inside. Single-angle lacing is also turned in and connected with the inside of the flanges.

All heavy members have vertical diaphragms every 5 or 6 ft. The diaphragms in all cases were planed to the exact width, and the sides of the members were assembled to them in their true position, the lacing being put on afterward. On account of this, in no case was any trouble caused by a twisting tendency while the members were being assembled, handled, or erected.

The trusses of the anchor arms are 64 ft. high at the hips (the U_2 points), 145 ft. 2½ in. over the main piers (the L_{10}-U_{10} post) and 62 ft. 0 in. at the ends of the cantilever arms (the U_{18} point) from center to center of chords. The suspended span is 57 ft. 0 in. deep,

and the approach span is 61 ft. 8 in. at the hips and 75 ft. 0 in. at the center, all from center to center of chords. In the cantilever bridge the trusses are 34 ft. 6 in. from center to center, and in the approach span they are 34 ft. 0 in.

Drawings and Proposals.

The Railroad Company furnished the specifications, the design, the stress sheets, and detail sheets developing each joint sufficiently to determine the construction, the shape, and the style of each member and all connections, for the purpose of estimating the weight of the material and the cost of fabrication and erection of the bridge.

Contractors were invited to submit alternate plans and proposals. None, however, was offered. The contract was awarded on the Railroad Company's design and specifications, and, in most of its details, was in the usual form. Profiting, however, by the experience of others in a recent disaster, it contained the following clauses which were actively operative.

"It is expressly understood and agreed between the parties hereto, that the Contractor shall be wholly responsible for the safe and satisfactory erection of the whole superstructure, and for this purpose the Contractor is required to, and does hereby expressly assume the responsibility for the correctness of the design and for the material being of good quality and properly fabricated, notwithstanding any previous inspection or acceptance of the material by the railroad.

"The Contractor shall execute and deliver to the Railroad, before any material is erected, its two bonds, satisfactory in form to the Railroad, and with such Surety Company as surety thereon as shall be approved by the Railroad. One of said bonds shall be conditioned for the faithful performance of this contract in so far as it applies to the fabrication, erection, and finishing of the cantilever span, and the other of said bonds shall be conditioned for the faithful performance of this contract in so far as it applies to the fabrication, erection, and finishing of the approach span. Each of the said bonds shall be in an amount equal to the total estimated payments which will be required by the Contractor from the Railroad before the completion of the respective spans. The Railroad will reimburse the Contractor in the amount of one-half of the premiums paid by it on said bonds."

The specifications, in most particulars, were not unusual, excepting in the eye-bar requirements, the substance of which is given elsewhere.

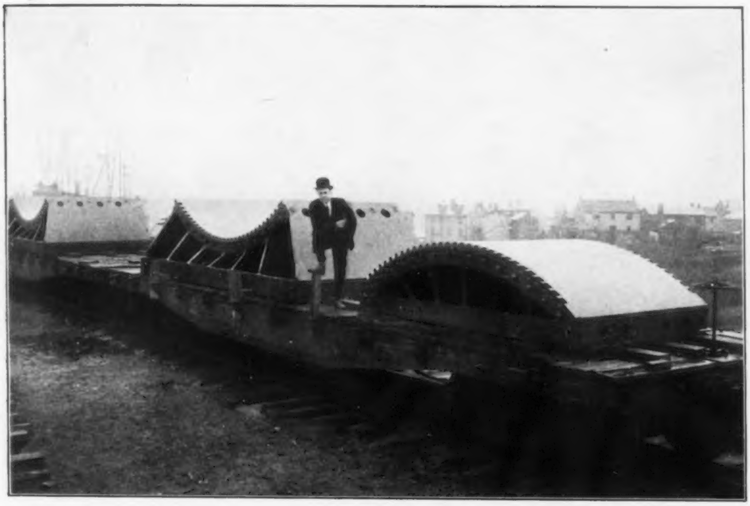


FIG. 1.—UPPER AND LOWER CASTINGS SUPPORTING TOWER POSTS.

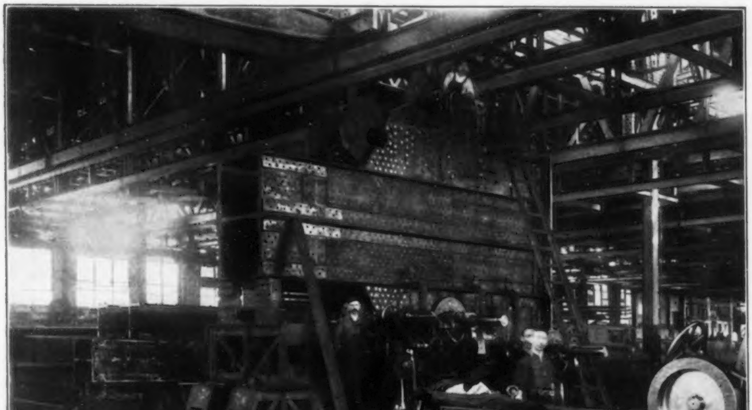


FIG. 2.—L_{10} "STUB," IN PROCESS OF MANUFACTURE.

Loading.

The floor system was proportioned for carrying two trains, one on each track, each weighing 6 000 lb. per lin. ft. and each preceded by two locomotives weighing 426 000 lb. each.

Two trains were assumed on each track wherever split loading produced greater results.

The trusses were designed for loads 10% less than these. The dead load averaged about 9 tons per lin. ft., of which ½ ton was for the timber deck and the rails. The dead-load stresses were computed from weights figured from the stress sheets. This method was repeated until the sections, computed on the basis of the actual dead load, figured from the shop drawings, came within 2% of the sections used.

The wind loads were proportioned by the areas of the exposed surfaces as finally developed, and were equivalent to about 1 100 lb. per lin. ft. of the lower chord for the anchor and the cantilever arms, about 1 000 lb. per lin. ft. for the lower chord of the suspended spans, and about 600 lb. per lin. ft. for the top chords. The wind loads were computed from the assumption of 300 lb. per lin. ft. on a train, and 30 lb. per sq. ft. on the exposed surfaces of the two trusses and the floor. Of this wind pressure, 10 lb. per sq. ft. was treated as being uniformly distributed over the whole structure, while the remaining 20 lb. was used as a moving load, applied wherever it produced the maximum effect on the members.

Unit Stresses.

The axial tension of the net section was assumed at 16 000 lb. per sq. in. The axial compression per square inch of gross section was assumed to be

$$\frac{16\,000}{\dfrac{1}{1 + \dfrac{L^2}{12\,000\,R^2}}},$$

but not to exceed 14 000 lb.

The impact allowed was:

$$I = L\,\frac{L}{D + L}$$

where I = Impact,
D = Dead load,
L = Live load,

excepting that impact in hangers was assumed to be 100% of L.

For wind stress, 20 000 lb. in tension was used, and a corresponding amount in compression, but not to exceed 18 000 lb. per sq. in.

For dead load, live load, impact, and wind load the units were 22 000 and 20 000 lb., respectively.

For erection stresses, 18 000 and 16 000 lb. for tension and compression, respectively, were assumed.

The pressure on concrete containing steel grillage was limited to 400 lb. per sq. in.

The bearing on the 12-in. rollers forming the bearings under the towers was limited to 6 000 lb. per lin. in.

In members subject to reversal of stresses, the resultant tensile stress and the resultant compression stress were determined, and each was increased by 80% of the smaller; the member was then designed so as to be capable of resisting either of the increased stresses. Shear in shop-driven rivets was taken at 11 000 lb. per sq. in. For hand-driven field rivets, the number was increased 25% over the number needed when shop-driven, and for hammer-driven rivets the number was increased 10 per cent.

Rivet bearing was figured at 22 000 lb. per sq. in., and bending on pins was taken at 24 000 lb., with corresponding reductions for field-driven rivets.

Anchorage and L_0 Details.

Under the specifications, the contractor was required to work out and furnish shop plans and details of all parts of the bridge structure. These details were extremely well designed, and the plans were very elaborate, in fact, one of the chief features of the construction of this work was the very unusual amount of attention given to the design of the details.

The anchorage stresses varied from 2 071 000 lb. compression per truss to 2 299 000 lb. tension. The L_0 pin point of the truss is 5 ft. 1¾ in. above the pin point at the upper ends of the eight 14 by 1½-in. eye-bars forming the anchorage for each truss. These pin points are connected by a rocker. The total motion of the L_0 point, from temperature and live-load changes, is 5 in., or 2½ in. on each side of the anchorage eye-bar pin center. This inclination (2½-in.) of the rocker out of the line of the anchor eye-bars produces, under extreme conditions, a horizontal thrust of about 100 000 lb., which, if not taken care of, would cause bending of the anchor eye-bars where

FIG. 1.—PUTTING IN PLACE THE PIN SUPPORTING DIAGONAL MEMBER.

FIG. 2.—PUTTING IN PLACE THE PIN SUPPORTING DIAGONAL MEMBER.

they leave the masonry at the bridge-seat level. To overcome this condition, a cast-steel jaw is bolted securely to each side of the lower part of the cast-steel bearing, which is anchored securely to the masonry, Fig. 1, Plate III. These jaws engage slides, one at each end of the pin in the upper ends of the anchor eye-bars. Under this arrangement, this pin can move vertically to any position desired, and at the same time the horizontal thrust from the rocker is transmitted through the pin to the slides, which find support against the jaws, thereby transmitting the stress to the masonry and entirely relieving the eye-bars from bending.

The rockers which connect the L_0 pins with the anchor eye-bar pins are solid links, designed to take care of the reversal of stresses mentioned and for rotation about the two 14-in. pins under load as the L_0 point travels backward and forward. This is accomplished by using cast-steel bushings lined with manganese-bronze around each pin.

The anchor eye-bar pins are supported in a cast-steel block resting on an adjustable wedge on the masonry bridge seat. This construction transmits the positive load from the L_0 point to the masonry, without putting any load on the anchor eye-bars. After the completion of the bridge, a special live load was placed on the suspended span and on the cantilever arm, in excess of the working live load, which condition produced an uplift on the anchor eye-bars in excess of that produced by working conditions, and, while the bars were thus elongated, the wedges under the eye-bar pin supports were forced tight, by driving and by special set-screws, and secured in this position. This design maintains a constant maximum tension in the anchor eye-bars. After the bars were placed under this stress, the openings in the concrete masonry around the bars were filled with soft concrete and sealed.

The anchorage in the masonry at the lower ends of the anchor eye-bars is made with large inverted shoes with 14-in. pin bearings.

I-beam grillage is built in the masonry just above these shoes to engage fully the masonry load; these inverted shoes were put in place as the masonry was being built; set-screws were used in leveling and adjusting them to exact lines and levels. Vertical shafts were formed in the concrete masonry in order to allow the eye-bars to be placed after the completion of the masonry. The lower pins were

entered ready for driving, and a horizontal passageway or tunnel was left at the lower pin elevation to allow access to the pins. A vertical manhole or shaft, about half way between the two sets of eye-bars, connected this tunnel with the bridge seat more than 50 ft. above.

Several months after the completion of the masonry, check measurements were made from the bridge seat down to the lower pins for ordering the eye-bars of exact length. The bars were put in place and the pins driven without any trouble.

Details of the Tower-Post Bearings on the Channel Piers, and of the L_{10} Points.

Originally, the bearings of the towers on the main piers were designed to be solid, without any rollers, and the contract was let with the plans calling for this construction. Considerable discussion and some criticisms were developed on this point at the time the various bridge companies were bidding on the work, and as a result, the Railroad Company furnished a revised plan, calling for radial roller bearings rotating around the intersection point of the lower chord with the tower posts—the L_{10} point—on a 9-ft. radius, with the object of reducing the secondary bending stresses from counter-flexure of the lower chords, and bending of the tower posts. This design delivers only vertical reactions centrally on the masonry piers. These bearings transmit to the masonry a loading of 12 558 000 lb., including live load, impact, and dead load. As a fixed bearing, calculations showed that secondary stress would be produced in the chord, L_9-L_{10}, ranging from $\pm$ 18 000 to $\mp$ 19 000 lb. per sq. in., and in the chord, L_{10}-L_{11}, ranging from $\pm$ 5 000 to $\mp$ 25 000 lb. per sq. in., and at the same time the masonry pressure was increased from an average of 400 to a maximum of 1 000 lb. per sq. in., which fully warranted the change in design as made, as an extra. This design reduced the secondary stresses in the chords mentioned to less than 3 000 lb. per sq. in. Each bearing consists of eighteen 24-in., 120-lb., I-beams, built flush in the concrete masonry for grillage. Sheet lead, $\frac{1}{4}$ in. thick, was used on this grillage under the cast-steel bearings. These castings, Fig. 2, Plate III, are in one piece, 18 ft. long and 8 ft. 6 in. wide, and weigh about 37 tons each. The tower-post is supported on an upper casting, and between the two are 29 segmental rollers, 4 in. wide, 12 in. in diameter, and 72 in. long. The upper casting is turned to a radius of 8 ft. 6 in., and the lower to 9 ft. 6 in. Care-

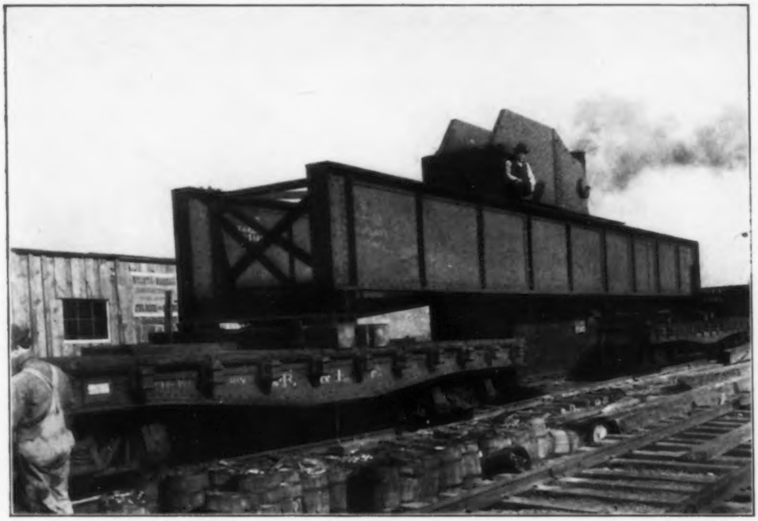


FIG. 1.—METHOD OF TRANSPORTING "STUBS."

FIG. 2.—SHOP ASSEMBLING.

ful inspection showed that these surfaces were machined correctly to within 0.002 in. A variation of 0.003 in. for low point was allowed on the rollers, but no point was allowed higher than the nominal diameter of the roller. The rollers are held in place by gear teeth at each end of each roller, and these engage in suitable racks bolted one to the upper and the other to the lower casting. By actual tests, made during erection and since, the motion of the rollers has been found to correspond closely with the estimated amount. The upper casting, Fig. 1, Plate IV, bearing directly on the rollers, weighs about 25 tons, and supports the tower post, which it fits and to which it is bolted.

The L_{10} stubs, Fig. 2, Plate IV, consisting of the L_{10} point with portions of the chord, L_9-L_{10} and L_{10}-L_{11}, and a portion of the bottom of the posts from the bearing on the castings to a splice point a short distance above the chords, were extremely difficult to make. In addition to the chords and the post being spliced solidly together, the plates, forming the L_{10} point, support by pins the bottoms of the diagonal members, U_8-L_{10}, and U_{12}-L_{10}, respectively, as shown on Plate V. These "stubs" had four planed bearing surfaces and two pin bearings which had to be located exactly right. These pieces, when finished ready for shipment, were 20 ft. 2 in. long, 15 ft. 10 in. deep, and 5 ft. 3 in. wide, and weighed 55 tons each. They were handled by rail, suspended between two 50-ft. girders, between the ends of cars supporting the ends of the girders, as shown by Fig. 1, Plate VI.

The sectional areas of the lower chords are spliced through from side to side of the L_{10} stubs, and the sections of tower posts above the chords are carried through the chords to the cast-steel bearings.

In this manner the horizontal and vertical stresses have their independent sections, and the whole mass is thoroughly connected by splice-plates with full shear values on all lines on which shears can develop. For erection purposes, the inclined members were connected to this point by pins.

Specifications.

The specifications applied generally, and were substantially adhered to.

Cast Steel.—Cast steel was required to show by specimen test from 65 000 to 70 000 lb. ultimate, with the elastic limit 50% of the

ultimate; the elongation in 2 in. to be 20%; the reduction of area 45%, and a cold-bending test of 90°, to a radius equal to the diameter of the test piece. No trouble was experienced in getting such results; the elongation was generally 25 per cent.

Cast steel requires an annealing temperature of 1 700° Fahr., and large pieces must be kept soaked at that temperature for from 5 to 6 hours and then must be kept covered so as to cool slowly and uniformly in from 48 to 60 hours, in order to insure a good result. Proper annealing is considered an essential in producing the desired result. Some of the pieces were condemned on account of the test pieces showing a coarse granular fracture; these were reannealed, after which the tests were entirely satisfactory.

The composition of the steel required to produce 70 000 lb. ultimate was about as follows:

Carbon	0.27 to 0.30
Manganese	0.65 to 0.70
Phosphorus	0.035
Sulphur	0.040
Silicon	0.27 to 0.30

Compression tests on cubes showed a low yield point. The first permanent set occurred at 13 000 lb.; the material commenced to flow at 120 000 lb., and at 250 000 lb. the cubes were reduced to about one-half their original height and the sides had increased from 1 in. to 1.43 in. without showing any fractures or other signs of breaking up.

The friction bushings of links, suspenders, and sliding bearing plates were of manganese-bronze. This material was required to show a test of not less than 75 000 lb. ultimate, not less than 35 000 lb. elastic limit in tension, and 28 000 lb. in compression, and not more than 10% set under a compression of 100 000 lb. per sq. in. It was also required to stand a cold-bending test through 120° to a radius equal to the thickness of the piece tested. The elongation required was not less than 25% in 2 in., and with a reduction of not less than 25% of the area. There was no difficulty in getting metal to meet these requirements.

The tests showed 80 000 lb. ultimate, and 42 000 lb. elastic limit, with compression close to the specification requirements. The mill-finished specimens would bend 180° and then spring back to 135 degrees.

PLATE VII.
PAPERS, AM. SOC. C. E.
JANUARY, 1911.
RAYMER ON
RAILROAD CANTILEVER BRIDGE.

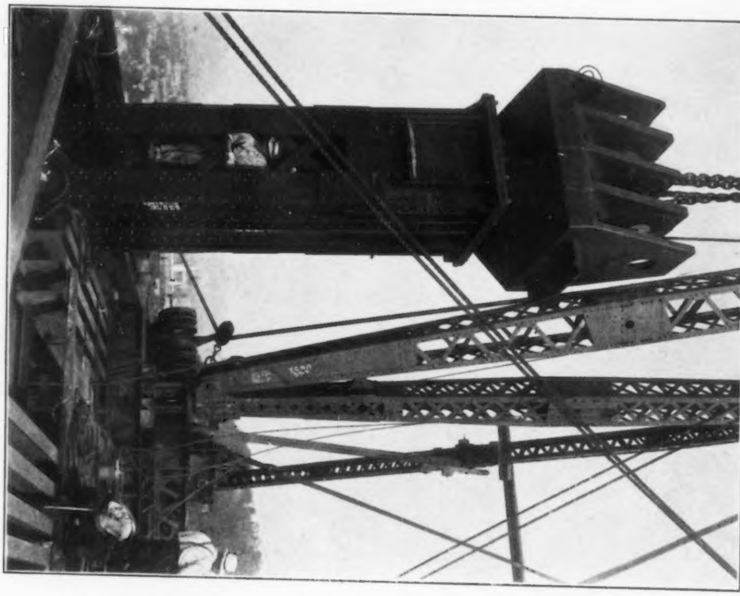


FIG. 1.—MAIN TOWER POSTS DURING ERECTION.

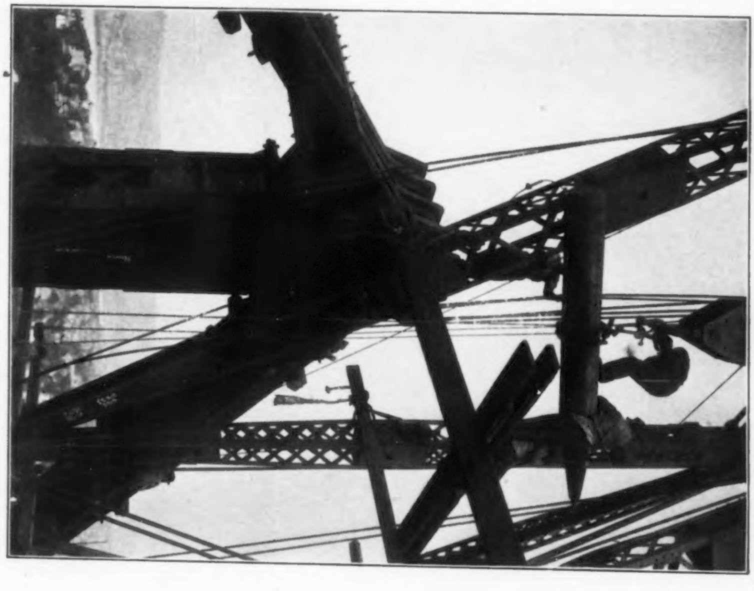


FIG. 2.—MAIN TOWER POSTS DURING ERECTION.

Eye-Bars.—The eye-bars were rolled at the Homestead Plant, from ingots which weighed from 9½ to 10 tons. From 35 to 40% was broken down and trimmed from the upper end, and from 10 to 15% from the lower end, of each ingot; that is, only about 50% of the ingot was used for eye-bar material. Only one of the largest bars came from one ingot, and, of the smaller bars, two came from one ingot. The slab which was used was rolled in the first heating to approximately twice the cross-section of the bar; it was then re-heated and finished to the proper size. The material was made into eye-bars at the plant of the American Bridge Company, at Ambridge, Pa., under the methods usually followed there. The bars were annealed by being heated slowly to a temperature of from 1 500 to 1 600° Fahr., and, when up to that temperature, were removed from the furnace and placed in a muffler open at each end; here they were allowed to cool until red heat was reached, then they were removed from the muffler and allowed to cool finally in the open air.

Eye-bar material was criticized only in a preliminary way by the specimen tests, which required it to be uniform and within certain limitations. The bars were accepted only as finished bars on the results of the representative full-sized tests.

Pins were drilled from end to end through their axes, and rough-turned before inspection and acceptance of the material.

The specifications also required all splices of members, and all joints and connections, to be shop-assembled in their correct positions, and while in that position all connection holes were reamed or drilled.

Details of the Main-Tower Posts, L_{10}-U_{10}, and of the U_{10} Casting.

The main-tower posts extend from the cast-steel bearing under the lower chord up to the cast-steel capping at the U_{10} point. This capping takes the reactions from the upper chord eye-bars, 16 of which, measuring 16 by 1⅞ in., are on each side of the 16-in. pin in the cast-steel cap. This casting, Plate VII, is in one piece, and weighs 37½ tons. It is provided with shelf-plates for the connection of the upper transverse strut on the axis of the pin, which forms part of the lateral bracing of the top chords. The main pin is 8 ft. 2½ in. long, and is supported on the cap by five vertical cast-steel ribs. The cap is 8 ft. 6 in. high, 7 ft. 6 in. wide, and 8 ft. 0 in. long, and is bolted to the top of the main post.

The main posts, L_{10}-U_{10}, have a cross-section of 640 sq. in. They are made up of a continuous diaphragm consisting of two $36\frac{1}{2}$ by $\frac{7}{8}$-in. web plates and four 8 by 8 by $\frac{3}{4}$-in. angles, three 66 by $\frac{7}{8}$-in. web plates on each side, with 8 by 6 by $\frac{7}{8}$-in. flange angles on the outside and 8 by 8 by $\frac{7}{8}$-in. flange angles on the inside. Central with the diaphragm, on the outside of the plates, are placed a $17\frac{1}{2}$ by $\frac{7}{8}$-in. plate and two 8 by 6 by $\frac{3}{4}$-in. angles, the 8-in. leg being against the main plate, and two 6 by $\frac{9}{16}$-in. fillers are between the outstanding 6-in. legs. Double $6\frac{1}{2}$ by $\frac{5}{8}$-in. lacing bars are used on the inside of the flange angles, and are tie-plated over the outer flange angles, similar to the chord construction. These posts were made up in three sections above the L_{10} stub. The total weight of each post is 224 tons.

Shop Assembling.

The main-tower posts, L_{10}-U_{10}, including the L_{10} stub, were assembled in the shop after the butting joints had been most carefully faced. The splices were drilled after the sections were in proper alignment and the joints had been drawn up to tight contact. In drilling these and other shop-assembled splices, the members were laid down with the inside face up. About every sixth hole of the splice or connection had been punched or drilled through all the material in order to allow assembling bolts to be used. All other holes were blank, except in the upper layer of plates or angles of each side, as it was lying in the shop, which was the outside layer of the inside web and the inside layer of the outside web. All holes were punched or drilled small, and, with these holes as a template, all the other holes were drilled through them; first short drills were used in the inner webs, then long drills were inserted through these holes and guided by them; these drills then entered the holes of the inner layer of plates or angles of the outside web, and were guided by them in passing through the remainder of the material. The holes, which had been previously provided all the way through the material for bolting, were reamed after the other holes were bolted up. When shop-assembling the trusses, the members were lined with a transit to the estimated dead-load deflection after erection.

After the posts, L_{10}-U_{10}, were drilled, the bottom chords, L_0-L_{10} and L_{10}-L_{18}, with the L_{10} stub, were assembled, Fig. 2, Plate VI, and properly lined up in a position corresponding to the deflection under dead load in the structure, in which position the splices were drilled,

TABLE 1.—Record of Full-Sized Eye-Bar Tests—Ohio Rive

No. of test.	Heat number.	Slab number.	Size of bar, in inches.	Elastic limit.	Ultimate strength.	Original area.	Area of fracture.	Red. of area, in %.	Original length, b. to b. pins.	Length after test, b. to b. pins.	Elongation, in per cent., b. to b. pins.	Original length of body.	Length of body after test.	Elongation of body, in per cent.	Location of break.	Elongation in 10 ft.
1	48 443	241 848	16 × 2⅛	38 690	67 170	34.24	23.46	31.5	37.54	43.40	15.60	31.00	36.07	16.30	17.62	1.87
2	41 006	1 784	16 × 2⅛	34 720	61 920	34.56	22.70	34.3	37.54	44.40	18.27	31 00	37.03	15.74	9.17	2.46
3	48 455	244 852	16 × 2⅛	36 500	65 760	34.52	28.32	18.0	44.36	51.74	16.63	38.00	44.61	17.00	5.40 to small area.	2.08
4	41 023	7 201	14 × 1½	34 130	65 500	21.68	11.91	45.0	35.03	41.52	18.20	29.00	34.85	20.10	11.77	2.43
5	48 445	242 340	14 × 1½	34 000	65 210	21.62	11.99	44.5	34.92	40.48	15.90	29.00	33.94	17.30	7.87	2.24
6	49 452	242 511	14 × 1½	35 140	64 480	21.34	13.02	38.9	41.07	47.52	15.70	34.00	40.84	20.10	41.60	2.28
7	41 013	3 817	16 × 2	33 060	60 200	32.36	15.80	51.1	37.58	45.09	15.90	31.00	37.64	21.40	5.65	2.84
8	41 008	2 311	16 × 2⅛	33 500	59 440	34.32	19.24	43.9	36.22	42.39	17.00	30.00	35.42	18.00	36.30	2.33
9	50 300	168 353	12 × 1¾	34 470	59 910	21.03	9.71	53.8	41.22	48.42	17.40	36.00	42.77	18.80	4.20	2.39
10	49 464	245 501	16 × 2	33 950	58 330	32.40	20.63	36.3	41.49	47.98	15.64	34.00	40.77	19.90	5.58	2.19
11	44 038	15 330	16 × 2	37 650	62 500	32.40	17.49	46.0	37.43	43.98	17.20	31.00	36.77	18.60	36.25	2.31
12	44 084	13 415	16 × 2⅛	36 400	64 720	34.61	18.53	46.4	37.46	43.80	17.20	31.00	36.63	18.10	35.80	2.58
13	44 039	15 865	16 × 1⅞	37 030	64 730	30.51	19.00	37.7	36.14	41.65	15.20	30.00	34.80	16.00	.....	1.96
14	44 009	2 635	16 × 1⅞	34 320	61 130	20.30	19.21	5.3	36.16	41.03	13.40	30.00	34.22	14.10	9.35	1.89
15	41 014	4 223	16 × 1⅞	37 920	63 090	30.59	17.49	42.8	36.11	42.76	18.40	30.00	35.89	19.60	6.00	2.63
16	49 462	244 916	16 × 1⅞	34 780	65 630	30.47	17.77	41.6	28.13	33.40	18.60	22.00	26.59	20.80	19.70	2.57
17	40 099	34 655	14 × 1⅝	33 360	62 860	23.24	15.24	34.4	34.93	40.18	15.00	28.00	32.51	16.10	10.00	1.91
18	44 073	26 584	14 × 1⅝	33 000	68 400	23.37	14.31	38.8	34.91	41.60	19.10	29.00	35.03	20.07	12.15	2.37
19	44 064	23 608	16 × 1¾	36 600	64 920	28.65	18.74	34.5	36.11	42.86	18.60	30.00	36.06	20.20	36.90	2.33
20	41 068	24 721	16 × 1¾	34 940	62 010	28.62	15.87	44.5	36.07	43.22	19.80	30.00	36.40	21.30	36.83	2.64
21	44 064	23 612	14 × 2	38 730	65 840	28.40	16.95	40.3	29.50	34.80	17.90	23.00	27.73	20.50	25.00	2.5
22	41 088	32 395	14 × 1 11/16	38 620	65 400	24.08	14.16	41.1	35.97	42.79	18.90	30.00	36.13	20.40	38.10	2.4
23	40 105	35 777	14 × 2	36 370	62 140	28.32	17.59	37.8	29.58	34.77	17.80	23.00	27.38	19.00	6.15	2.3
24	42 108	45 183	14 × 1 11/16	36 030	62 440	24.42	13.91	43.0	30.20	36.40	20.50	25.00	30.64	22 60	11.60	2.5
25	41 061	21 524	14 × 1⅝	35 620	65 450	23.30	11.59	50.1	29.68	34.95	17.40	24.00	28.67	19.40	29.95	2.3
26	41 084	30 779	14 × 1⅝	37 660	66 720	23.23	11.59	50.1	29.90	35.70	19.40	24.00	29.15	21.40	30.63	2.5
27	40 067	22 823	16 × 2⅛	34 180	64 180	33.64	22.61	32.7	33.24	39.42	18.60	27.00	32.48	20.20	5.54	2.3
28	41 063	22 015	16 × 1 11/16	36 020	74 440	27.20	15.60	42.6	33.34	38.26	14.70	27.00	31.31	15.90	6.15	2.2
29	44 060	21 577	16 × 1 11/16	36 760	66 170	27.20	19.18	29.4	33.26	39.18	17.80	27.00	32.20	19.20	5.45	2.2
30	40 078	26 523	12 × 1⅝	35 480	64 460	20.01	10.67	46.6	28.59	34.09	19.20	24.00	29.02	20.90	19.00	2.4
31	33 076	....	10 × 1 5/16	35 000	63 840	13.00	6.31	51.4	28.85	33.81	17.10	24.00	28.57	19.00	30.10	2.1
32	40 106	36 291	14 × 1⅞	34 580	62 970	26.60	14.04	47.2	29.52	35.23	19.30	24.00	29.08	21.10	7.00	2.5
33	34 079	....	8 × 1⅝	36 320	66 400	12.80	7.68	40.0	41.04	45.74	....	28.00–"*A*" 30.00–"*B*"	32.10–"*A*" 3.82–"*B*"	27.30–"*A*" 10.70–"*B*"	40.25 from "*A*"	...
34	34 079	....	8 × 1⅝	34 140	67 180	12.80	Broke in head	..	41.07	45.16	....	28.00–"*A*" 5.00–"*B*"	30.57–"*A*" 5.32–"*B*"	9.10–"*A*" 6.40–"*B*"	In head "*B*"	...
35	50 296	166 611	12 × 1½	30 500	58 510	18.03	10.34	42.6	40.93	47.76	16.60	35.00	41.24	17.80	8.15	2.4
36	45 235	161 569	12 × 1¾	30 800	61 610	21.10	11.35	46.2	40.94	47.53	16.10	35.00	41.07	17.30	41.00	2.4
37	45 237	162 279	12 × 1¾	35 390	64 560	20.91	10.60	49.3	41.07	50.15	22.10	35.00	43.52	24.30	38.03	2.9
38	45 237	162 280	12 × 1½	34 620	65 060	18.06	9.63	46.6	40.85	48.56	18.87	35.00	42.10	20.20	34.23	2.5
39	45 234	161 282	14 × 1⅝	35 300	65 870	22.77	13.10	42.4	31.81	37.71	18.50	26.00	31.20	20.00	27.80	2.5
40	45 238	162 782	14 × 1⅝	34 640	67 960	22.80	18.58	18.5	31.81	36.47	14.60	26.00	30.02	15.40	8.40	1.5
41	46 294	169 337	12 × 1¾	35 980	64 910	21.26	17.09	19.6	41.05	46.96	14.30	36.00	41.55	15.40	13.54	1.5
42	42 286	....	8 × 1⅜	36 800	59 900	10.85–"*A*" 10.90–"*B*"	Broke in head "*B*"	..	40.82	44.99	....	27.00–"*A*" 6.00–"*B*"	29.36–"*A*" 6.47–"*B*"	....	Head "*B*"	0.9
43	42 286	....	8 × 1⅜	*L* 33 170 *S* 32 800	62 210 61 500	10.85 10.96	Broke in head "*A*"	..	41.00	45.10	....	19.00–"*A*" 15.00–"*B*"	20.88–"*A*" 16.50–"*B*"	....	Head "*A*"	1.0
44	41 011	3 121	16 × 2	29 850	64 320	32.16	18.33	43.0	33.24	38.52	19.40	26.00	30.40	17.30	3.12	2.2
45	41 011	3 099	16 × 2	39 270	67 330	32.08	18.46	42.4	37.54	44.22	17.70	31.00	36.83	18.80	7.83	2.4
46	41 086	31 744	14 × 1 11/16	29 290	61 500	23.90	12.76	46.6	30.84	36.83	16.61	25.00	30.31	25.24	20.90	2.6
47	41 086	31 730	14 × 1 11/16	37 410	64 620	23.52	14.21	39.5	36.00	42.42	17.00	30.00	35.71	19.00	36.50	2.3
48	41 063	22 009	16 × 2⅛	34 230	65 550	34.32	21.05	38.6	33.32	38.50	15.54	27.00	31.46	16.51	5.68	2.0
49	40 066	22 400	16 × 2⅛	31 600	61 460	34.49	23.42	32.1	33.26	38.73	16.44	27.00	31.81	17.81	5.28	2.1

s—OHIO RIVER BRIDGE, PITTSBURG AND LAKE ERIE RAILROAD, 1909.

Location of break.	Elongation in 10 ft.	Per cent. elongation in 10 ft.	Fracture.	Specimen Tests: Unannealed.				Specimen Tests: Annealed.				Chemical Analysis.				Member.	Contract number.	Remarks.
				El. Lim.	Ult. Str.	El. Lim.	Ult. Str.	El. Lim.	Ult. Str.	El. Lim.	Ult. Str.	Car.	Phos.	Mn.	S.			
17.62	1.87	18.70	Granular........	36 540	69 130	37 970	68 100	36 960	62 470	35 400	62 900	0.24	0.024	0.65	0.032	U_8-U_9	3 599	
9.17	2.46	24.60	Silky............	38 360	68 180	37 770	65 440	36 860	61 920	36 380	57 810	0.28	0.010	0.53	0.030	U_8-U_9	3 599	
5.40 to small area.	2.08	20.80	None......	38 850	70 420	38 100	66 800	35 950	63 360	36 520	63 360	0.27	0.015	0.55	0.034	U_8-M_7	3 599	Did not break.
11.77	2.43	24.30	Silky Irreg......	36 410	71 780	38 500	67 160	36 620	66 120	37 450	65 820	0.24	0.015	0.64	0.030	L_{0A}-L_{0B}	3 599	
7.87	2.24	22.40	Silky Irreg......	37 950	70 580	38 950	69 740	35 690	58 740	36 140	59 800	0.24	0.030	0.65	0.031	L_{0A}-L_{0D}	3 599	
41.60	2.28	22.80	Silky......	38 760	66 780	37 640	67 180	34 760	58 090	35 600	63 360	0.28	0.020	0.61	0.033	L_{0A}-L_{0C}	3 599	
5.65	2.84	28.40	90% Silky......	44 760	66 060	42 810	64 640	36 210	57 610	35 720	57 670	0.25	0.015	0.55	0.023	U_8-U_9	3 599	
36.30	2.33	23.30	Silky Irreg......	35 960	63 400	38 120	66 560	35 670	59 020	36 230	57 980	0.25	0.014	0.53	0.028	U_5-M_5	3 599	
4.20	2.39	23.90	Silky Ang.......	40 350	65 920	40 860	66 980	36 500	58 160	37 040	59 660	0.24	0.015	0.59	0.028	U_2-M_3	3 599	
5.58	2.19	21.90	Silky............	38 450	64 120	37 690	64 170	35 530	57 450	35 940	58 340	0.25	0.012	0.56	0.031	L_6-M_7	3 599	
36.25	2.31	23.10	Silky ½ Cup.....	39 890	65 300	38 760	67 820	35 320	60 600	37 180	60 980	0.24	0.019	0.65	0.025	U_{11}-U_{12}	3 599	
35.80	2.58	25.80	Silky Ang.......	37 800	69 180	38 420	65 780	37 080	62 620	36 800	61 880	0.24	0.030	0.65	0.025	U_{11}-U_{12}	3 599	
.....	1.96	19.60	Silky............	39 160	67 920	38 430	66 820	34 900	60 870	33 380	59 290	0.27	0.011	0.61	0.032	U_{10}-U_9	3 599	
9.35	1.89	18.90	Silky............	35 440	56 900	35 710	57 000	38 570	67 340	37 900	65 960	0.25	0.018	0.56	0.030	U_{10}-U_{11}	3 599	
6.00	2.63	26.30	Silky Ang.......	39 460	66 040	40 510	66 530	35 500	58 960	37 010	56 860	0.25	0.014	0.56	0.030	U_{10}-U_9	3 599	
19.70	2.57	25.70	Silky Irreg......	38 980	66 890	38 780	67 040	37 000	61 940	36 690	60 180	0.28	0.012	0.60	0.031	U_{10}-U_{11}	3 599	
10.00	1.91	19.10	Silky............	38 210	68 160	38 820	68 320	35 420	60 900	34 780	60 060	0.26	0.010	0.53	0.027	U_{13}-U_{14}	3 601	
12.15	2.37	23.70	Silky............	40 190	71 800	41 410	72 480	36 320	59 160	35 870	64 140	0.27	0.018	0.62	0.028	U_{13}-U_{14}	3 601	
36.90	2.33	23.30	Silky Ang.......	38 370	67 980	37 670	67 540	35 000	60 040	35 640	56 840	0.26	0.012	0.63	0.030	U_{14}-M_{15}	3 601	
36.83	2.64	26.40	Silky Cup	38 310	65 640	37 770	64 800	34 910	61 560	34 040	62 490	0.27	0.016	0.65	0.026	U_{12}-M_{13}	3 601	
25.00	2.53	25.30	Silky Irreg	38 370	67 980	37 670	67 540	35 000	60 040	35 640	56 840	0.26	0.012	0.63	0.030	U_{19}-U_{20}	3 601	
38.10	2.41	24.10	Silky Ang.......	38 090	68 480	38 780	67 020	34 810	64 040	35 400	63 120	0.24	0.028	0.60	0.031	U_{12}-U_{13}	3 601	
6.15	2.30	23.00	Silky............	39 150	67 010	39 550	68 480	35 300	60 060	35 480	60 820	0.25	0.022	0.50	0.030	U_{18}-U_{19}	3 601	
11.60	2.56	25.60	Silky Ang	37 190	65 520	37 530	65 420	35 830	61 040	35 700	60 320	0.27	0.010	0.59	0.024	U_{15}-U_{16}	3 601	
29.95	2.33	23.30	Silky ½ Cup.....	38 690	68 120	39 300	69 860	34 470	63 180	35 390	65 180	0.25	0.027	0.57	0.030	U_{17}-U_{18}	3 601	
30.63	2.53	25.30	Silky Cup.......	38 520	68 000	37 780	62 970	35 000	61 720	35 960	61 700	0.24	0.030	0.65	0.030	U_{16}-U_{17}	3 601	
5.54	2.30	23.00	Silky............	38 200	66 660	38 090	65 040	35 220	59 620	34 240	56 830	0.20	0.012	0.62	0.030	L_{14}-M_{13}	3 601	
6.15	2.26	22.60	Silky Ang.......	40 560	73 980	40 560	71 960	35 740	62 260	36 240	61 660	0.27	0.026	0.67	0.025	L_{16}-M_{15}	3 601	
5.45	2.24	22.40	Silky Irreg......	38 950	71 800	38 870	69 660	35 540	65 080	35 260	68 920	0.27	0.029	0.62	0.025	L_{16}-M_{15}	3 601	
19.00	2.40	24.00	Silky Ang.......	38 550	67 620	38 170	65 880	26 340	58 350	35 180	58 330	0.28	0.019	0.60	0.026	L_{22}-M_{21}	3 603	
30.10	2.15	21.50	Silky............	41 490	67 490	43 370	67 090	39 220	62 100	40 620	63 760	0.26	0.027	0.63	0.032	U_{22}-M_{23}	3 603	
7.00	2.50	25.00	Silky Irreg......	39 040	65 760	38 970	67 140	35 460	56 900	35 500	59 960	0.25	0.020	0.58	0.030	L_{22}-L_{23}	3 603	
40.25 from "A"	....	......	....	42 130	69 580	40 740	68 070	39 850	63 940	39 820	62 950	0.30	0.025	0.53	0.025	M_{21}-U_{22}	3 603	
In head "B"	....	......	....	42 130	69 580	40 740	68 070	39 850	63 940	39 820	62 950	0.30	0.025	0.53	0.025	M_{23}-L_{22}	3 603	
8.15	2.47	24.70	Silky............	39 160	66 300	39 460	64 700	35 000	60 780	33 660	61 240	0.27	0.011	0.51	0.022	M_5-L_6	3 607	
41.00	2.47	24.70	Silky 80%......	43 900	65 440	38 790	64 060	32 530	57 570	31 290	62 240	0.25	0.010	0.60	0.024	U_2-M_3	3 607	
38.03	2.94	29.40	Silky 90%......	49 470	69 200	42 650	69 000	37 190	61 740	34 140	59 020	0.26	0.019	0.64	0.030	U_2-M_3	3 607	
34.23	2.51	25.10	Silky Ang.......	49 470	69 200	42 650	69 000	37 190	61 740	34 140	59 020	0.26	0.019	0.64	0.030	M_5-L_6	3 607	
27.80	2.50	25.00	Silky 90%......	43 270	72 420	41 160	70 500	36 020	65 720	35 340	62 510	0.27	0.020	0.60	0.028	L_5-L_6	3 607	
8.40	1.51	15.10	Silky............	44 220	71 730	42 700	69 330	35 390	64 120	33 790	62 120	0.24	0.021	0.65	0.034	L_5-L_6	3 607	
13.54	1.58	15.80	Granular........	40 520	68 420	41 020	69 200	35 800	62 310	35 860	63 550	0.27	0.010	0.53	0.040	U_2-M_3	3 607	
. Head "B"	0.90	9.00	80% Silky Gran.	40 620	62 260	......	......	41 020	65 520	......	......	0.29	0.026	0.50	0.012	M_5-U_4	3 607	
. Head "A"	1.02	10.20	80% Silky Gran.	40 620	62 260	......	......	41 020	65 520	......	......	0.29	0.026	0.50	0.012	L_4-L_5	3 607	
3.12	2.22	22.20	Silky............	38 070	72 400	35 650	69 850	34 480	61 560	36 150	57 100	0.28	0.021	0.62	0.030	L_7-M_7	3 599	
7.33	2.47	24.70	Silky............	38 070	72 400	35 650	69 850	34 480	61 560	36 150	57 100	0.28	0.021	0.62	0.030	U_8-U_9	3 599	Re-test.
20.90	2.67	26.70	Silky............	38 250	67 030	38 200	65 630	35 720	57 190	34 950	60 270	0.26	0.012	0.55	0.049	U_{14}-U_{15}	3 601	
36.50	2.33	23.30	Silky Ang.......	38 250	67 030	38 200	65 630	35 720	57 190	34 950	60 270	0.26	0.012	0.55	0.049	U_{12}-U_{13}	3 601	Re-test.
5.68	2.06	20.60	Silky Cup.......	40 560	73 980	40 560	71 240	35 740	68 960	36 240	61 680	0.27	0.026	0.65	0.025	L_{14}-M_{13}	3 601	
5.28	2.17	21.70	80% Silky......	37 720	65 710	38 140	65 830	33 600	61 200	34 820	59 500	0.27	0.025	0.68	0.029	L_{14}-M_{13}	3 601	

as described for the main posts. The bottom chords, from L_0 to L_8, were then, by bending, put in a position to correspond to the position of these chords in the truss without stress; the top chords and web members up to L_6-U_6 were then assembled to these chords where necessary. The web members were bent so that when the truss was swung under dead load their alignment would be straight. After the truss was lined up accurately and made true in every part, all the connections were drilled as described for the tower posts.

For the cantilever arm, the bottom chord, L_{12} to L_{16}, was assembled to the members, L_{14}-U_{14} and L_{16}-U_{18}, in the deflected position, after which the connections at L_{14} and L_{16} were drilled.

As it was considered undesirable to depend on bolts during erection, the members, L_{14}-U_{14} and L_{16}-U_{18}, were field-spliced near the L_{14} and L_{16} points. After the splices in these members had been properly drilled and tightly bolted up, they were assembled to the bottom chord, as described; and, after the connections to this chord had been drilled, the splices in the members were cut and the short pieces were riveted to the bottom-chord panel points without changing their position from that in which they were reamed or drilled. This did away with any dependence on field bolts during erection, as perfect butt joints were available; it also insured having the connections made in a position which corresponded with the final deflected position of the truss. When coupling up during erection, these members had to be bent; this, however, caused no trouble.

Floor-beam and stringer connections were generally sub-punched and reamed to templates.

Inspection and Tests.

The work at all times, at the mill, the shop, and in the field, was under thorough inspection. All measurements were made by the contractors, and also by the inspector, with steel tapes and other instruments carefully compared with the Railroad Company's standards, which had been checked and corrected by the Bureau of Standards at Washington, D. C. Steel-tape measurements were made under uniform tension by the use of spring balances.

The unusually large riveted connections and splices, built up of many thicknesses of plates and angles, presented difficulties in providing perfectly flat plates of even thickness.

The chief reason that the results, as to the quality of material

and workmanship, were so satisfactory, apart from the great interest everyone responsible for any part showed in the work, was the fact that the material and the work were in a certain sense inspected before the work was done. This was accomplished by the very efficient manner in which the inspectors discussed in advance all details (and particularly the difficult points to be overcome) with the mill men and the shop men; in this manner hearty co-operation was secured, which insured satisfactory results.

Some of the joints required flat plates and rolled shapes to be built up to a thickness of 7¼ in. Any variations in the thickness of these plates or in their flatness would have caused serious trouble.

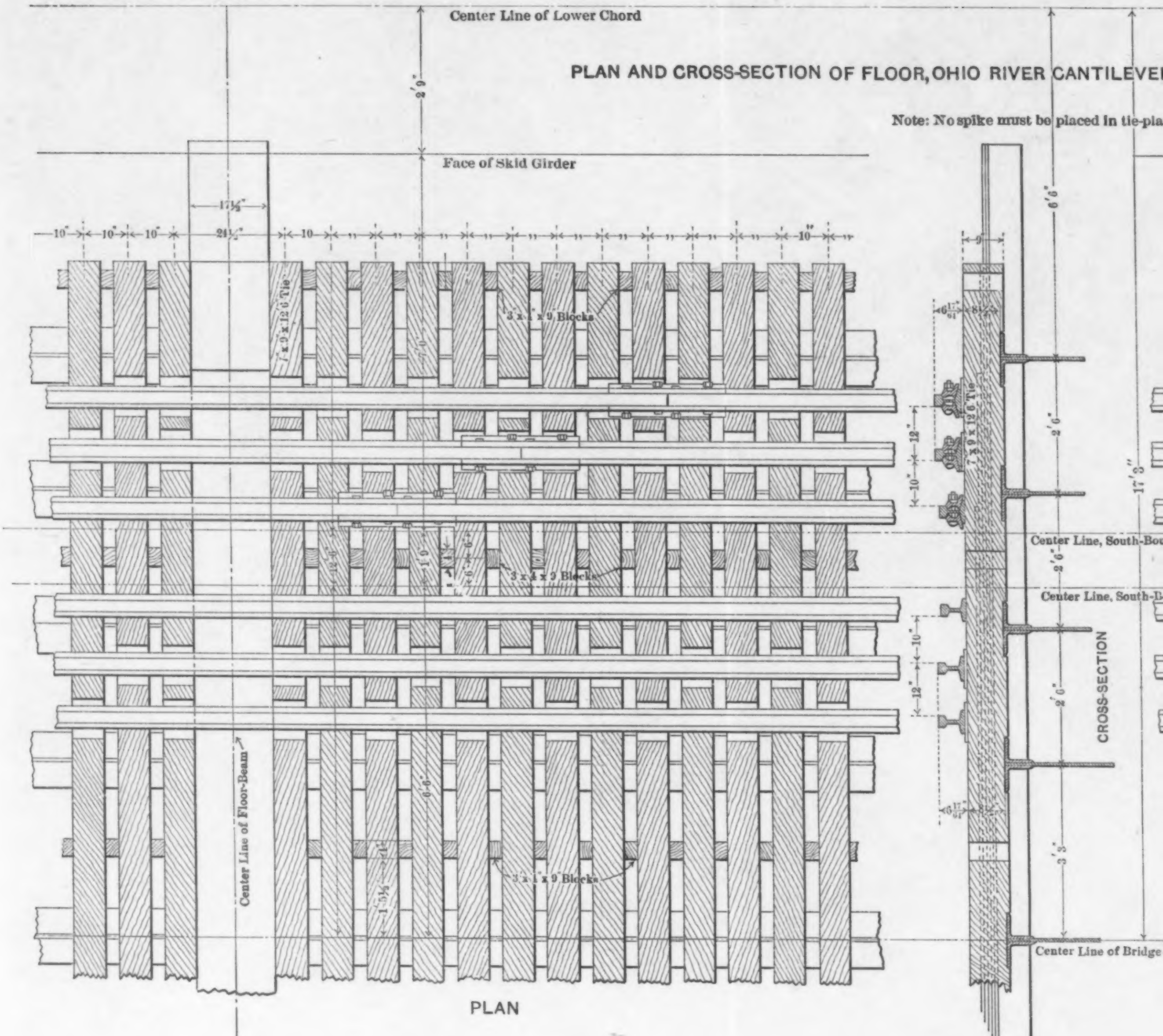
The work of driving 1-in. rivets through this thickness of material, in the shop and in the field, so that the rivet would be tight, also required much study and care by the contractors as well as by the inspectors. The results secured were entirely satisfactory. The best shape for long rivets in the stock near the head was determined by much experimental driving. Milling and boring were done by taking a rough cut first, then the measurements were checked and adjustments made if necessary before making the finishing cut. All pin holes were located very carefully for position and direction. Eye-bar pin holes were bored with two cuts. The clearance between the pins and the pin holes, for pins from 12 to 16 in. in diameter, was required to be $\frac{1}{32}$ in., and the variations were limited to from 0.03 to 0.04 in. In order to meet this narrow margin, every cutter was carefully gauged and recorded, and every pin hole cut by each cutter was measured and recorded. It was found that the holes from 12 to 16 in. in diameter, made by all cutters, were uniformly from 0.004 to 0.005 in. larger than the size of the cutter. After this was established there was very little trouble in cutting the pin holes to the exact diameter desired. The cutters had to be changed frequently, on account of wear. A few pin holes had been made before the variations in size between the pin and the hole had been determined; some of these, with variations of 0.024 in., caused considerable trouble during erection, as the pins drove exceedingly hard.

All material was tested, and many tests were made for compression. Tests were also made on rollers, on sections 1 in. long.

Full-sized compression members were not tested. Full-sized eye-bars were tested, as required by the specifications. Table 1, Plate VIII, gives the results of these tests.

PLAN AND CROSS-SECTION OF FLOOR, OHIO RIVER CANTILEVE

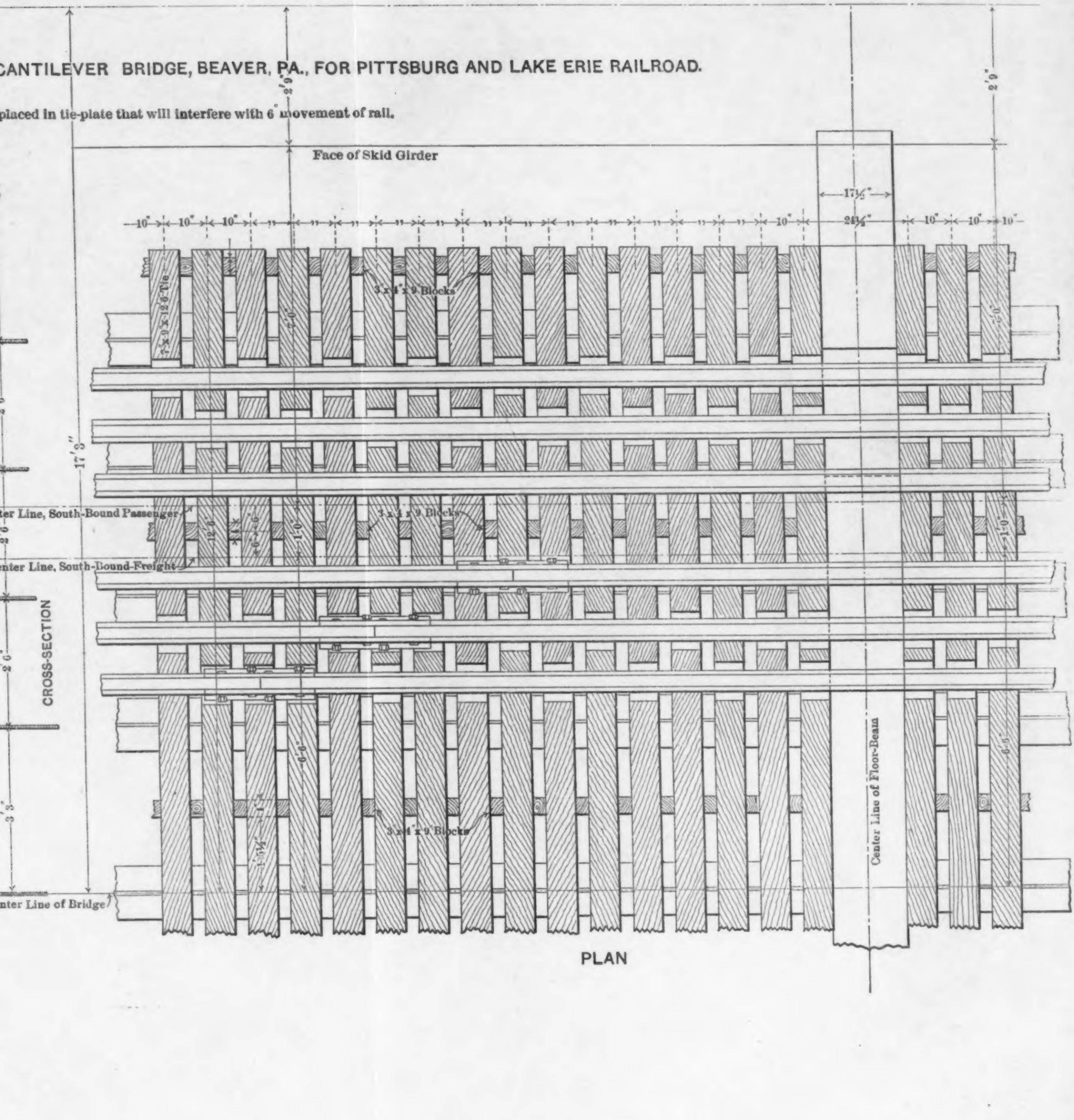
Note: No spike must be placed in tie-pla



PLAN

CANTILEVER BRIDGE, BEAVER, PA., FOR PITTSBURG AND LAKE ERIE RAILROAD.

placed in tie-plate that will interfere with 6" movement of rail.



PLAN

The annealing of eye-bars and steel castings received much attention; the eye-bar tests show great uniformity, and the thorough annealing of the steel castings made the steel in them very ductile.

Eye-bars were tested for elastic limit, ultimate strength, stretch, and reduction. The stretch of the bar due to the bedding of the pin against the eye was noted; and, although it could not be correctly measured, it was found to be generally as much as $\frac{1}{8}$ in. for each head when the elastic limit was reached.

Rolled-steel coupons, 1 in. in diameter and 1 in. long, taken from eye-bar stock, showed a yield point in compression of from 27 000 to 29 000 lb. per sq. in., at which time the first permanent set of from 0.001 to 0.002 in. could be observed. The yield increased to 15% at 100 000-lb. pressure. After the 100 000-lb. pressure was passed, the steel flowed, the test pieces swelling in diameter under the slowly increasing pressure until, at 250 000 lb., the original height of 1 in. was reduced 50% and the original diameter of 1 in. was increased to $1\frac{7}{16}$ in. This pressure did not develop any distortions or surface cracks.

The 12-in. roller sections were tested in pieces 1 in. thick cut from the ends of the rollers. This test was made to develop the safe bearing load per linear inch of the 12-in. rollers under the tower posts. The loads applied to specimens of this size varied from 10 000 to 16 000 lb. by 1 000-lb. increments. No set could be detected, after these loads had been applied in turn; at 25 000 lb., one specimen showed a set of 0.006 in.; at 32 000 lb., another showed a set of 0.007 in.; at 50 000 lb., another showed a set of 0.032 in.

The first two tests given were made on a compression testing machine, and were satisfactory; the last one was made on a tension machine, which did not work properly.

The maximum loading the 12-in. rollers will have to support in the bridge will be less than 6 000 lb. per lin. in.

In summing up the results of the tests, it appears that rolled medium steel, of 66 000 lb. ultimate strength and 35 000 lb. elastic limit in tension, has a yield point in compression of from 27 000 to 29 000 lb.; this steel sets 15% under a compression of 100 000 lb. per sq. in.

Cast steel of from 65 000 to 70 000 lb. ultimate strength and 35 000 lb. elastic limit in tension has a yield point in compression of about 13 000 lb., and sets 16% under 100 000 lb. on 1-in. cylinders. Manganese-bronze of 80 000 lb. ultimate strength and from 41 000

to 43 000 lb. elastic limit in tension, has a yield point in compression at 28 000 lb. per sq. in., and a set of from $9\frac{1}{2}$ to 10% under a load of 100 000 lb.

Riveting.

The material was punched $\frac{1}{8}$ in. smaller and reamed $\frac{1}{16}$ in. larger than the nominal diameter of the rivet, with the exception of laterals, skid girders, lacing, tie-plates, and diaphragms, in which members full-sized punching was allowed; material more than $\frac{3}{4}$ in. thick, in tension, and more than $\frac{7}{8}$ in. thick, in compression, was drilled from the solid. No punching was allowed in the big connection plates in the trusses. When assembling the joints, made up of many thicknesses of plates and angles, the material was cleaned and painted with much care, the paint used on the surfaces in contact being made thin in order that the thickness might not be increased appreciably. In the trusses of the anchor and the cantilever arms, in the floor-beams, and also in the connections between stringers and floor-beams, generally, 1-in. rivets were used, and $\frac{7}{8}$-in. rivets in the other parts. All 1-in. rivets with more than 4-in. grip were tapered, these rivets having a diameter of $1\frac{3}{64}$ in. under the shop head and 1 in. at the point. The tapered rivets had to be forced into the holes with a slight pressure. They filled the holes admirably, and gave excellent results in the shop and in the field. The largest grip used was $7\frac{1}{2}$ in. The number of loose field rivets was less than 2 per cent. The field rivets were driven with pneumatic hammers. Numerous tests showed that straight 1-in. rivets, with a grip of $4\frac{1}{2}$ in. or more, bent sidewise in the hole about 4 in. from the unfinished head, and, after coming to a bearing on the walls of the rivet hole, all upsetting below this point ceased.

Tapered rivets stood up straight and filled the holes. The points of all long rivets were dipped in water before driving. It was found necessary to have the diameter of the rivets not greater than $1\frac{3}{64}$ in. at the base of the taper. For this reason the dies for making these rivets had to be renewed frequently. If the rivets were too large, the walls of the rivet hole would scrape the metal back, and a fin would be formed between the shop head and the member, making it impossible to get the rivet tight.

Skid Girders.

The skid girders, two on each side of the bridge, are unique and of unusual interest, in forming, with the special floor design, a

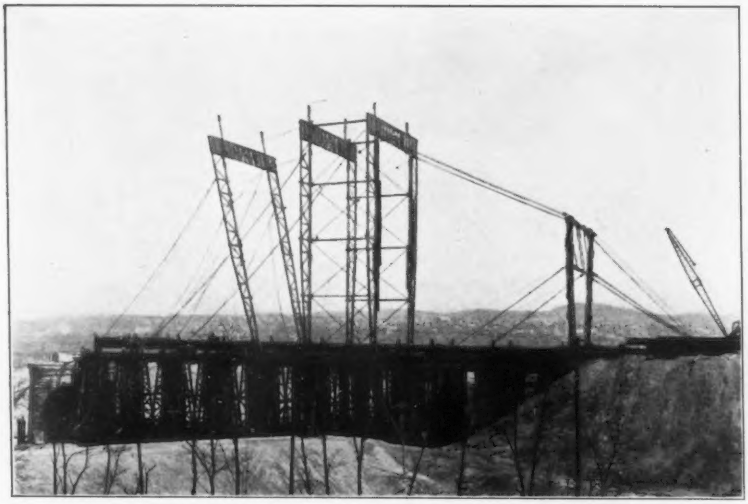


FIG. 1.—ERECTION OF TRAVELER.

FIG. 2.—ERECTION OF ANCHOR ARM.

remarkable protection against a catastrophe in case of a derailment. There are two horizontal skid girders for each truss, one vertically over the other, 4 and 6 ft., respectively, above the top of the ties. The chords on the track side of the trusses are made up of 6 by 6 by ½-in. angles and 12½ by ½-in. web plates. The outside chords have 6 by 6 by ½-in. angles and 10½ by ½-in. plates. These chords are 5 ft. 4½ in. apart, and are connected by web members made up of 3½ by 3½ by ⅜-in. angles.

The track faces of these skid girders are smooth and continuous, and 7 ft. 0 in. from the center line of the adjacent track. They have expansion joints at the proper locations, with rivets countersunk on the track faces of the chords.

The skid girders are attached to the main-truss members by angles connecting with the skid-girder chord web-plates.

DETAILS.

Especial attention was given to the details of the chord splices and the connections of the web members with the chords, so as to pick up the material gradually and also have the splice material arranged symmetrically as far as was possible.

In the anchor arm the panel points are shop-riveted to the bottom-chord sections. It was decided to be undesirable to locate the splices in the bottom chord of the cantilever arm in this manner, on account of the great weight, for erection purposes, of a chord section combined with a riveted panel point, which weight would be about 86 tons. For this reason, splices were made in the bottom chords on each side of the main panel points, thereby reducing the weight of the heaviest single member to about 49 tons.

The suspended span is hung on independent pins; the upper one is located in the attachment plates of U_{18} joint, vertically below the panel-point pin; the lower one is in the attachment plates connecting the truss members at the end of the suspended span, and directly above their intersection point, and at the same time not in the way of ample floor-beam connections. This arrangement simplified the erection problems and permitted the use of a very satisfactory development of bushing around the pins in the suspension hangers so as to reduce the pin-bearing stresses to values suitable for the motion of rotation under loads.

The suspension members are built up of stiff sections, and are proportioned for the loads in tension as well as for bending stresses caused by a friction of 25% on the pins resisting rotation.

The pin holes in these members are provided with steel bushings and manganese-bronze linings. Eye-bars were considered, but were not used, as the estimated effect of the bending stress was too great.

The floor-beams are riveted to the trusses, and the stringers are riveted to the floor-beams only in alternate bays; at all other points they are supported on shelves, with room for expansion at the free ends, so as to limit the secondary stresses in the floor system and the bending of the free ends of the floor-beams due to changes in the length of the chords under live-load stresses.

FLOOR.

The floor on this bridge was designed to be safe, not only when all wheels are on the rails, but also under any probable condition resulting from a derailment on the bridge.

Four stringers, 2½ ft. from center to center, are placed under each gauntlet, and there is a ninth line of stringers under the center line between the trusses to support the inner ends of the ties. Each gauntlet has four main-track rails and two inner-rail guard-rails, making twelve lines of 100-lb. rails on the bridge floor.

The ties are of 7 by 9-in. white oak, 12½ ft. long, placed on edge and sized to 8½ in. The inner ends of the ties butt on the center line of the center stringers. The ties are 3 in. apart in the clear. This spacing is maintained by using three lines of wooden-block spacers or separators on each half of the floor, to prevent bunching.

The tops of the ties are about 1½ in. above the tops of the floor-beams, and are held about ¼ in. away from them by the use of metal plates, in order to allow cinders to be blown off the floor-beams by passing trains.

Wooden guards are not used, as the modern steel-car construction, with drop doors, has forced their location below a plane through the top of the rails within 5 ft. of the center of the track, thereby defeating the object of their use. The spacing of the ties, formerly secured in an indifferent manner by dapping the guard-rails over the ties, is accomplished much more effectively by the block spacers used.

All the ties and the spacing blocks, oak and pine, respectively, have been treated chemically by immersion in an open tank of Carbolineum

FIG. 1.—ERECTION OF ANCHOR ARM.

FIG. 2.--CREEPER TRAVELER ERECTING CANTILEVER ARM.

Avenarius heated to a temperature of 260° Fahr. The construction of the deck is shown on Plate IX. All track rails are supported on double-shoulder tie-plates secured to the ties by screw-spikes, as shown on Figs. 1 and 2. No special expansion joints are used in the rails.

The bridge expansion provision amounts to about 5 in. at the L_0 points and to a total of 8 in. at the ends of the suspended span.

The rails are 33 ft. long. The standard rail splices used allow an extreme movement of $\frac{3}{8}$ in. at each rail joint, which amount is sufficient to accommodate a temperature change of 140° Fahr. The rails were given the proper spacing when laid, with the assumption that the temperature would vary from 20° below to 120° above zero. The rails were then immediately secured to the floor over the "fixed" points of the bridge, and, on the shore side of the abutment, on the tracks approaching the bridge, by the use of efficient anti-creeping devices, as shown on Fig. 3. Between these fixed points the rails are free to travel on the tie-plates. All spikes in the vicinity of the joints are located so that they will not interfere with the free travel of the rails, the assumption being that both the rails and the bridge are provided with sufficient freedom to expand and contract between these fixed points, the rails by distributed expansion points, and the bridge at one or two located points; and that the two movements will be adjusted by the rails moving between the fixed points in the tie-plates. No anti-creeping devices are used on the suspended span.

Wind.

Considerable wind stresses have to be taken care of by the anchor masonry at the L_0 points, where, under the passage of each train, the vertical reactions are zero, and the normal reactions under dead loads alone are not great. For these reasons, it was decided to transfer the wind stresses to the anchor masonry by using wind buffers built into the masonry and securely anchored to it. These buffers engage with the bridge between brackets attached to the bottom of the end floor-beams, bearing plates of manganese-bronze being used between the buffers and the brackets. These buffers allow freedom of motion in the bridge, longitudinally and vertically, but resist all side motion.

The wind bracing, commencing at these buffers, is continuous between the upper as well as the lower chords to the U_{20} point in the upper system and to the L_{18} point in the lower, where it engages the

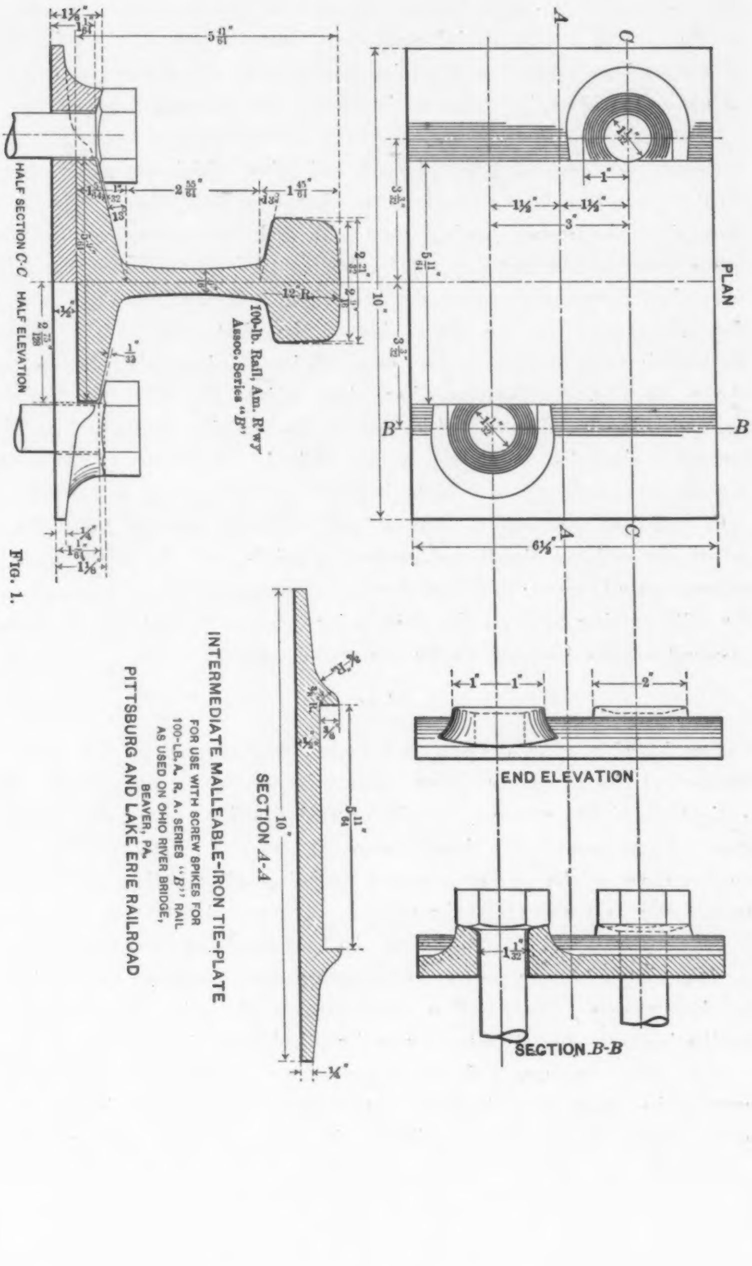


FIG. 1.

INTERMEDIATE MALLEABLE-IRON TIE-PLATE
FOR USE WITH SCREW SPIKES FOR 100-LB. A. R. A. SERIES "B" RAIL AS USED ON OHIO RIVER BRIDGE, BEAVER, PA.
PITTSBURG AND LAKE ERIE RAILROAD

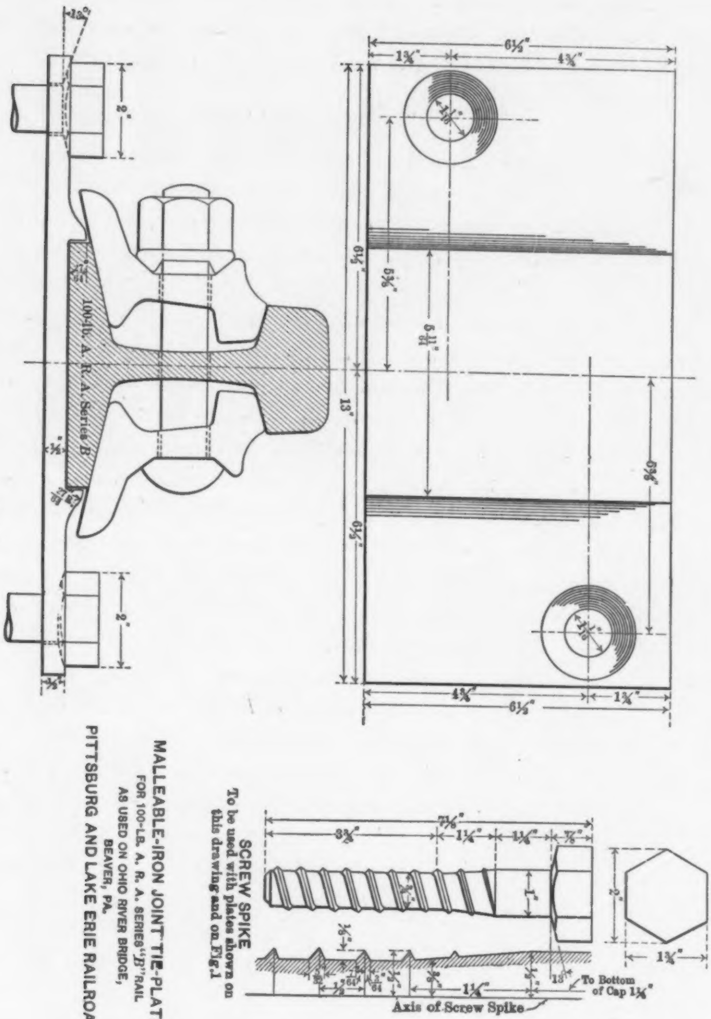


FIG. 2.

MALLEABLE-IRON JOINT TIE-PLATE
FOR 100-LB. A. R. A. SERIES "B" RAIL
AS USED ON OHIO RIVER BRIDGE,
BEAVER, PA.
PITTSBURG AND LAKE ERIE RAILROAD

portal strut of the suspended span and its end floor-beam, respectively, at their middle points, by pins and castings sliding between bronze plates in the strut, or between the brackets on the end floor-beam. The wind bracing in the suspended span is complete between these supporting points at its ends. All wind bracing is stiff throughout.

Falsework and Erection.

The falsework was designed to support heavy concentrations of load under the anchor arms, making double bents necessary under the main panel points, and was built on pile foundations, the piles being driven to refusal by a steam hammer. No settlements occurred in the falsework during erection.

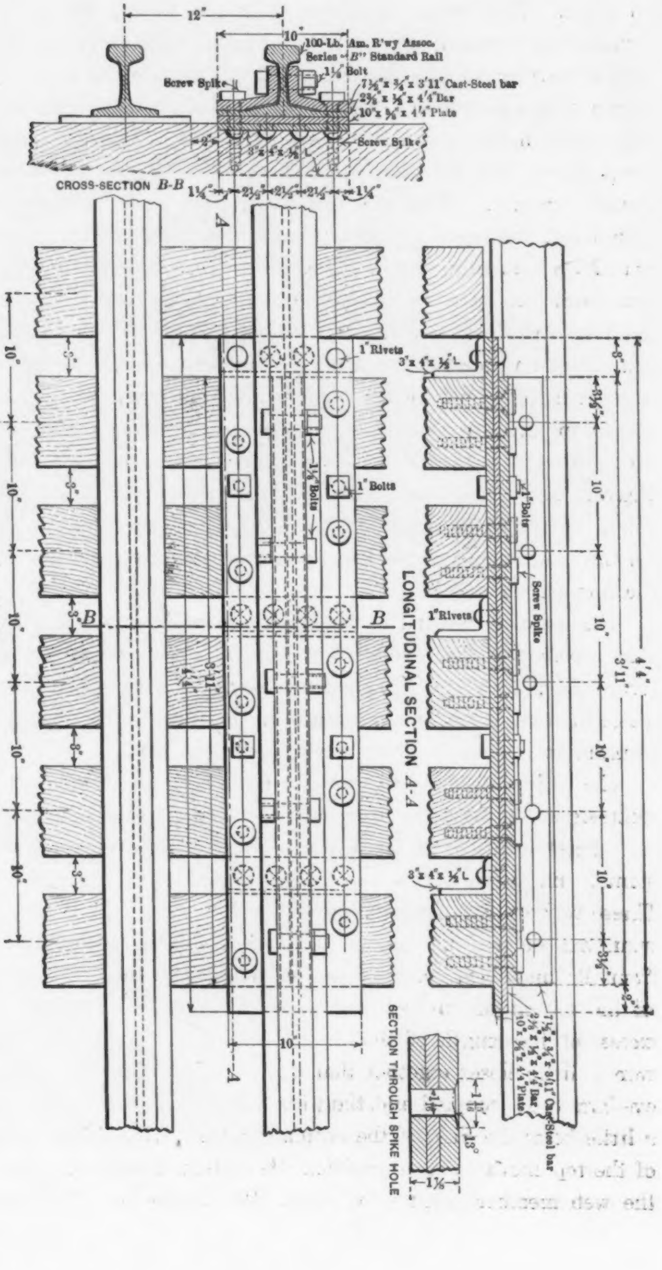
The work of erection was handled very efficiently and expeditiously by the contractor. An outside gantry traveler was used for all material erected on the falsework; this included the fixed span, the anchor arms, and a sufficient portion of the cantilever arms beyond the main piers to allow the erection of the main towers, the inclined posts, and the first two-panel section of the lower chords of the cantilever arms, as shown by Figs. 1 and 2, Plate X, and Fig. 1, Plate XI. The remaining portions of the cantilever arms and the suspended span were erected with a creeper traveler (as shown by Fig. 2, Plate XI, and Figs. 1 and 2, Plate XII), or a traveling derrick, attached to the top-chord panel points when in operation, and moved ahead on ways attached to the same points, which points were designed to serve this purpose.

The creeper traveler was very complete in all its details, its weight was counterbalanced while moving it from panel to panel, and it was handled with great certainty and promptness. The erection was commenced at the south or Monaca abutment by placing the anchor eyebars and the L_0 steel castings. The floor-beams and stringers were then put in place to receive the material tracks. This work was continued to the L_{10} point, where the cast-steel tower-post bearings and rollers were put in place.

The "L_{10} stubs" were then put in position on the cast-steel bearings. These stubs consisted of the lower sections of the posts, with strut sections of the lower chords and the L_{10} panel point, with all its plates and angles shop-riveted. This work was done with the gantry traveler. The traveler was then moved toward the abutment, setting the lower-chord sections, and attaching them to the floor-beams as they were put

Fig. 1.—Erecting Cantilever Arm with Creeper Traveler.

Fig. 2.—Erection of Cantilever Arm.



PLAN

DESIGN FOR ANTI-CREEPING RAIL DEVICE, PITTSBURG AND LAKE ERIE RAILROAD

FIG. 8.

in place. This being completed at the L_0 point, the erection of the trusses was commenced at the same point, and the work was finished as the traveler was moved from point to point to the main-tower posts, where it was placed in a practically central position over the pier. In this position the towers were erected, the upper sections and the U_{10} posts being put in place by stiff-leg derricks placed on top of the gantry traveler. With the derricks in this position, the towers were completed, the upper chords and the first double-panel sections of the cantilever arm were put in place, and all connections for extending the cantilever arm were coupled in the completed L_{12} and U_{12} points. These derricks were then used in erecting the creeper traveler in place on the completed top chords between the U_{11} and U_{12} points. The gantry traveler was practically built in the structure as the tower sections were completed up through it, making it necessary to take the traveler down in order to remove it. When this was done, the traveler was moved to the Beaver end of the bridge, and used in the erection of the 370-ft. fixed span, as shown by Fig. 1, Plate XIII, after which it was moved forward and used on the Beaver anchor arm erection, the same as at the Monaca end.

The material erected by the gantry traveler was delivered by cars; that erected by the creeper traveler was delivered by barges on the river (Fig. 2, Plate XIII), and hoisted directly from them. The material was loaded on the barges at the contractors' bridge plant, at Rankin, Pa., about 35 miles above the bridge site.

The adjusting device used during erection, in making the final connections between the two halves of the suspended span, consisted of wedges operated by large screws in the bottom chords at the L_{18} points, and toggle joints in the top chords, also operated by screws. These wedges and toggles were set so that each half of the suspended span was erected and held in a position a little forward and upward from its final location, making the distance between the erected ends of the top chords on each side of the center of the bridge a little in excess of the length of the closure section. The bottom chord points were a little closer together than their final position, which allowed the eye-bars to be coupled and the pins driven with the center panel point a little below the line of the bottom chords. After the closure sections of the top chords were in position, the bottom chords connected up, and the web members secured in place, the wedges and the toggles were

FIG. 1.—ERECTING FIXED SPAN WITH TRAVELER.

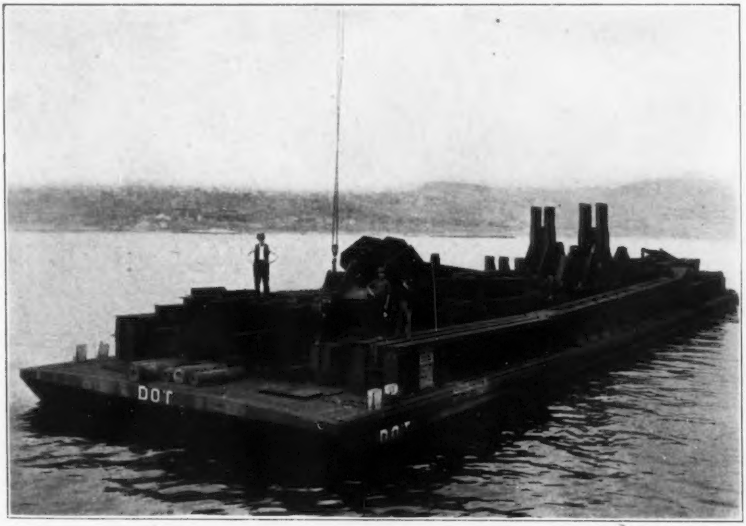


FIG. 2.—BRIDGE MATERIAL DELIVERED ON BARGES.

operated, and, as the two parts of the suspended span came into position, the chord splices were connected, thereby completing the span This closure is shown on Plate XIV.

The mode of erecting the bridge, and the devices, derricks, and tools necessary for erection and adjustment, also the detail of the falsework under the anchor arms and the fixed span were left to the choice of the contractor, subject to the approval of the Railroad Company as to their general sufficiency and strength.

Weights.

Steel anchors....................	399 368 lb.
Trusses, floors, and skid girders....	31 441 211 "
Cast-steel work, including main-tower rollers..................	526 211 "
Total..................	32 366 790 lb. = 16 184 tons.

The 370-ft. fixed span included in the foregoing figures weighs 2 206 tons, and the suspended span, also included in the total, weighs 1 596 tons. The dead load in the wooden deck and rails amounts to 900 tons.

Special Loading on the Bridge Used to Adjust the Wedges at the L_0 Points.

After the completion of the bridge, and before it was put in service, a special loading was placed on it in order to develop stresses in the anchor eye-bars in excess of the working loads, and thereby allow the wedges at the L_0 points to be set up so as to maintain this maximum tension in these eye-bars. For this purpose, eight locomotives and twenty loaded cars were made up in two trains, one for each track, each consisting of five cars, four locomotives, and five cars, weighing as follows, for each track:

5 cars	749 100 lb.
4 locomotives	1 344 000 "
5 cars	749 100 "

These trains were placed side by side, with the forward ends at the tower pier adjacent to the anchorage to be adjusted, the trains being entirely on the cantilever arm and the suspended span.

Elevations were taken at each floor-beam before, during, and after the application of the load, and the "travel" of the U_{10} point was noted

by using a transit to plumb down to the masonry from that point. The details of the measurements are given in Table 2.

TABLE 2.—ELEVATIONS OF FLOOR-BEAMS ON OHIO RIVER BRIDGE DURING TEST, MAY 10TH, 1910.

Point read.	West Side.				East Side.			
	Before loading.	During load.	Difference.	After load removed.	Before loading.	During load.	Difference.	After load removed.
North Span (Load on North Cantilever Arm and Suspended Span).								
L 0.......	755.659	755.695	—0.036	755.667	755.653	755.669	—0.016	755.675
L 4.......	756.043	756.115	—0.072	756.049	756.038	756.109	—0.071	756.050
L 6.......	756.176	756.245	—0.069	756.181	756.163	756.234	—0.071	756.170
L 10.......	756.405	756.397	—0.006	756.401	756.398	756.397	—0.001	756.396
L 14.......	756.455	756.262	—0.193	756.444	756.460	756.272	—0.188	756.448
L 16.......	756.457	756.155	—0.302	756.441	756.460	756.160	—0.300	756.435
L 18.......	756.430	756.028	—0.402	756.409	756.435	756.016	—0.419	756.407
L 23.......	756.416	755.986	—0.430	756.388	756.412	755.953	—0.459	756.383
South Span (Load on South Cantilever Arm and Suspended Span).								
L 23.......	756.411	755.955	—0.456	756.396	756.423	755.956	—0.467	756.392
L 18.......	756.451	756.031	—0.420	756.444	756.456	756.043	—0.413	756.428
L 16.......	756.461	756.148	—0.313	756.457	756.475	756.180	—0.295	756.460
L 14.......	756.481	756.263	—0.218	756.475	756.479	756.292	—0.187	756.465
L 10.......	....	756.397	.....	756.403	....	756.412	.....	756.410
L 6.......	756.197	756.221	—0.024	756.194	756.203	756.261	—0.058	756.200
L 4.......	756.065	756.088	—0.023	756.064	756.064	756.121	—0.057	756.065
L 0.......	755.679	755.688	—0.009	755.680	755.676	755.676	—0.000	755.674
North Span (Load on Anchor Span).								
L 0.......	755.659	755.666	—0.007	755.667	755.653	755.654	—0.001	755.675
L 4.......	756.043	755.974	—0.069	756.049	756.038	755.981	—0.057	756.050
L 6.......	756.176	756.116	—0.060	756.181	756.163	756.111	—0.052	756.170
L 10.......	756.405	756.389	—0.016	756.401	756.398	756.389	—0.009	756.396
L 14.......	756.455	756.456	—0.001	756.444	756.460	756.464	—0.004	756.448
L 16.......	756.457	756.462	—0.005	756.441	756.460	756.464	—0.004	756.435
L 18.......	756.430	756.449	—0.019	756.409	756.435	756.441	—0.006	756.407

Top of North Tower moved 0.183 ft. S. under load.
" " South " " 0.150 " N. " "
" " North " " 0.030 " N. " " on Anchor Span.

The deflection of the bridge and the movement of the U_{10} points corresponded almost exactly with the estimated amounts. The rollers at the L_{10} point also responded to the deflection in the lower chord, at the rates of the radius of the roller bearing to the height of the tower.

Under the auspices of Mr. Howard, of the Bureau of Standards, of Washington, D. C., extensive measurements are being made with instruments of precision between points drilled in the material. These

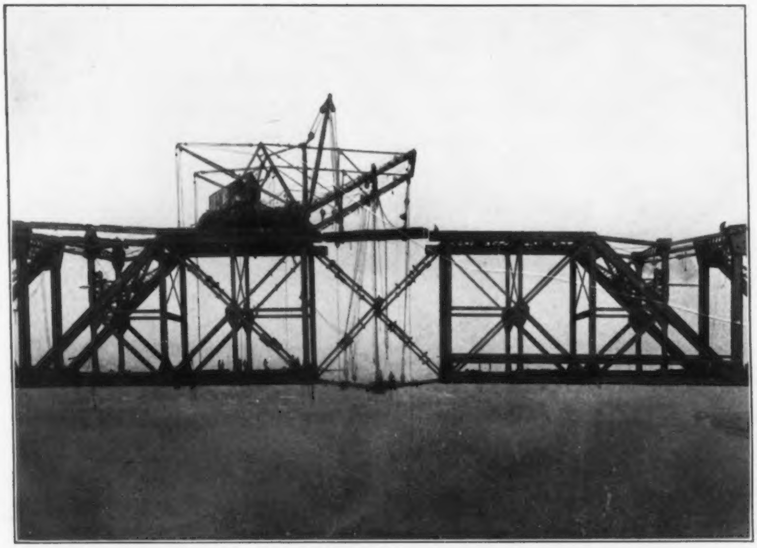


FIG. 1.—CLOSURE OF SUSPENDED SPAN.

FIG. 2.—CLOSURE OF SUSPENDED SPAN.

points are approximately 20 in. apart, and measurements were made while the members were under no stress before erection. Similar measurements are being made now—after erection—to determine, by the variations in length between the holes, the amount of dead-load stress in the members. These measurements are precise enough to determine variations of from 150 to 200 lb. per sq. in., and it is hoped that a complete report of them will be made by the Bureau of Standards in the near future. By this method it has been determined that the dead-load stresses actually existing in the lower chords at each side of the main-tower posts are, on an average, 5 800 lb. for the L_8-L_{10} members and 5 700 lb. for the L_{10}-L_{12} members, and that the total stresses under the live loading noted amounted to 10 000 lb. per sq. in. in the same members.

This bridge was constructed for The Pittsburg and Lake Erie Railroad Company, Colonel J. M. Schoonmaker, Vice-President, J. A. Atwood, M. Am. Soc. C. E., Chief Engineer, A. R. Raymer, M. Am. Soc. C. E., Assistant Chief Engineer, and Albert Lucius, M. Am. Soc. C. E., Consulting Bridge Engineer and Inspector.

It was built by the McClintic-Marshall Construction Company, of Pittsburg, Pa., Paul L. Wölfel, M. Am. Soc. C. E., Chief Engineer, and was completed in May, 1910.

The writer is under obligations to Messrs. Lucius and Wölfel for valuable assistance in the preparation of this paper.

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PAPERS AND DISCUSSIONS

DAMS ON SAND FOUNDATIONS: SOME PRINCIPLES INVOLVED IN THEIR DESIGN, AND THE LAW GOVERNING THE DEPTH OF PENETRATION REQUIRED FOR SHEET-PILING.

By Arnold C. Koenig, Assoc. M. Am. Soc. C. E.
To be Presented March 15th, 1911.

All textbooks on the subject of dams which the writer has had occasion to examine emphatically discourage the idea of attempting to construct permanent dams on any kind of foundation other than solid rock bottom, although most of them contain one or more illustrations of the types which are generally designed for construction on alluvial soils, such as firm beds of clay, or deep strata of cemented gravel (commonly called "hardpan"), or on sand, provided the conditions are such as to permit the construction of a curtain, or cut-off wall, of sheet-piling, or a core-wall of masonry or of clay puddle, so as to penetrate a firm stratum of denser material.

The frequent failures of sheet-pile dams, on the "bottomless" sand beds of Western streams, have served merely as the basis for warnings, and not as incentives to study and investigation, on the part of the authors of the standard books on the subject of dams, which the practising engineer consults for information and precedents.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Many crude brush and log dams were constructed, and maintained successfully for long periods of years, on these "bottomless" sand beds, by practical and resourceful pioneers who had no theories or precedents other than the example of the beaver.

The operations of British engineers in contending with sand, during the construction of the large Government irrigation works in India, furnish some expensive precedents for the expenditure of unlimited funds; but, our pioneer Western irrigation surveyors were confronted with the problem of coping with streams having apparently bottomless beds of shifting sands, with funds which were generally limited to the few dollars which the pioneer settlers could borrow on their personal characters and future prospects, because their "desert" homesteads, which now rank among the finest farms in the country, were in those times practically worthless as collateral security.

The writer has spent most of his life in the sandy regions of the West, and has had occasion to seek information on practical methods of dealing with treacherous sand, as a young engineer whose ambitions were involved in the successful solution of such problems; therefore after several years of professional familiarity with it, the term, "quicksand," has less terror for him than it ordinarily arouses.

The following paper is submitted for the consideration of those who have occasion to deal with such problems, not in the sense of establishing authoritative rules, but with the hope of arousing discussion, from which it may be the writer's privilege to learn things which have not been brought out in his own experience.

Seepage, Percolation, and the Movement of Underground Waters.—The velocities of flow which occur in seepage, percolation, or in the forced flow of water through sand, gravel, clay, loam, or other pervious materials, are affected by coefficients of capillary attraction and frictional resistance.

The effects of these forces are co-existent, but not identical, and they vary inversely as the sizes and proportion of the interstices between the particles of the alluvium. In fine clay, therefore, in which the interstices are of infinitesimal size, it is almost impossible to force an appreciable flow of water through a thickness of a few feet. If water finds passage, however, through any point of variable density, or any fissure or shrinkage crack, a barely appreciable trickle, under pressure, will rapidly disintegrate and abrade the clay, and, as the

stream thus increases in volume, will quickly undermine or destroy any structure which may be dependent on the density of the clay for its stability.

In the case of sand or gravel, with relatively large interstices, the forces of friction and capillary attraction offer less resistance, and therefore water may be forced through them with an appreciable velocity of flow.

In any alluvial soil in contact with, or subject to the pressure of, water, a movement of water through it is inevitable, whether it be the infinitesimal progress of seepage through fine clay, percolation through sand, the flow through an embankment of broken stone, or the rush of water through the crevices and open spaces between large boulders, the safe, permissible velocity of flow must be confined to that which the material will stand without the movement, displacement, or abrasion of the particles which compose it.

Turneaure and Russell* refer to numerous experiments which have been carried out at various times to determine the velocity at which water will flow through sands and gravels of different grades of fineness. These experiments were made for various purposes, such as to determine the dimensions of sand filters, the probable supply of water from wells, etc.

D'Arcy, Hagen, Hazen, and others, found that for grades of sand varying from very fine sand to fine gravel, the velocity of flow closely followed the law of flow through capillary tubes.

In considering the flow of seepage, or ground-waters, we are not concerned in refinements of experimental data which indicate the influence of temperature on the rate of flow; therefore, after eliminating these, the result of the experiments is the following formula:

$$V = 8\,200\ d^2 s.$$

In which V = the velocity of flow, in feet per day;

d = the effective size of the sand grains (varying from 0.1 mm., which is very fine sand, to 3 mm., which is fine gravel); and

s = the slope of the ground-water surface (hydraulic gradient).

According to this formula, the velocity of flow of ground-water,

* In "Public Water Supplies."

through fine sand having an effective size of grains of 0.1 mm. and a general slope of ground-water surface of 5 ft. per mile (hydraulic gradient = 0.001), would be:

$$V = 8\,200\ d^2 s = 8\,200 \times 0.01 \times 0.001 = 0.082 \text{ ft. per day.}$$

The velocity of flow through gravel having an effective diameter of grains of 3 mm. and a general slope of water surface of approximately 50 ft. per mile (hydraulic gradient = 0.01) by this formula would be:

$$V = 8\,200\ d^2 s = 8\,200 \times 9 \times 0.01 = 738 \text{ ft. per day.}$$
$$= \text{approximately } 0.0085 \text{ ft. per sec.}$$

The Report of the Massachusetts State Board of Health for 1902 contains the results of a series of experiments to determine the velocities of flow of water which it is possible to obtain through screened gravel, of various grades of fineness, assuming 40% porosity. These results are shown in Table 1.

TABLE 1.—Velocity of Flow of Water, in Feet per Day in Screened Gravel, Assuming 40% Porosity, Based on Experiments of the Massachusetts State Board of Health.

Slope, *s*.	Effective size, in millimeters.									
	3	5	8	10	15	20	25	30	35	40
0.0005	28	82	164	246	410	656	902	1 230	1 640	2 050
0.001	57	172	335	475	820	1 210	1 680	2 250	3 030	3 690
0.002	115	328	639	902	1 550	2 250	3 030	3 930	4 830	5 820
0.004	221	631	1 230	1 700	2 870	3 930	5 000	6 060	7 130	8 200
0.006	336	918	1 690	2 250	3 690	5 080	6 390	7 620	8 930	10 100
0.008	443	1 160	2 060	2 780	4 340	5 900	7 380	8 930	10 400	11 800
0.01	549	1 410	2 460	3 150	5 000	6 800	8 440	10 000	11 500	......

From an examination of the figures in Table 1, it becomes evident that all the foregoing experimental data are, at best, only rough approximations. The formula: $V = 8\,200\ d^2 s$, which was determined from experiments with various grades of sand, gives much larger results than the quantities in Table 1 for screened gravel. To modify the formula so as to produce the results given in that table would involve a variation of the coefficient (8 200) between 6 222 (for $d = 3$ mm. and $s = 0.0005$) to 940 (for $d = 35$ mm. and $s = 0.01$).

For many years the water supply of the Citizens Water Company, of Denver, was secured from about a mile of timber crib, 30 in.

square in section, and about a mile of perforated 30-in. pipe, both submerged in the sands of the Platte River at depths varying from 14 to 22 ft. The timber crib was open at the bottom, so that, with the openings in the cribbing, perhaps one-half of its superficial area was open for the inflow of water, while, for the pipe line, the net area of the perforations would probably bear a much smaller ratio to the circumferential area of the pipe. It may be assumed, therefore, that on the 2 miles of combined crib and pipe line the total area of inlet openings would approximate 25% of the surface area exposed to contact with the water-bearing sand, thus affording a total net area of inlet openings of 26 000 sq. ft. The water was led away through pipes, by natural flow, and the supply thus secured was between 400 000 and 450 000 cu. ft. per day, which is equivalent to 13 cu. ft. per day per square foot of inlet opening, or a velocity of inflow of 0.00015 ft. per sec.

Being submerged under the bed of the river at depths varying from 14 to 22 ft., these cribs are entirely surrounded by water-bearing sand; therefore, as the water may approach radially from every direction, it must be assumed that the maximum velocity of flow through the sand is less than one-fourth of the velocity of inflow through the perforations and openings.

Assuming an average head of 16 ft. as acting on these cribs, the spouting velocity of water entering the pipe through one of the perforations from a free body of water under the pressure due to a 16-ft. head ($V = \sqrt{2\,g\,h}$) would be 32 ft. per sec., while the actual average velocity of inflow is only $\frac{1}{200\,000}$ of such rate.

All formulas for the velocity of flow of water, under special conditions, are modifications of $V = \sqrt{2\,g\,h}$, with coefficients introduced to provide for the special conditions of frictional resistances encountered. In working backward from the results indicated above, so as to determine the special coefficient which must be applied to the formula to adapt it to this special case, we obtain the following:

$$V = 0.00004\ \sqrt{h}.$$

The lack of harmony in the results indicated in all the foregoing data makes it impractical to devise even an approximate formula for the determination of the magnitude of the force produced by the flow of seepage water under a line of sheet-piling. Even if it were possible to determine the relative effects of frictional resistance and capillary

attraction on the flow of water through interstices of such an infinite variety as is afforded by the range of materials from fine clay to large boulders, such "hair-splitting" refinements of calculation would have very little practical value for this purpose, on account of the uncertainties which attend the determination of the variations of actual conditions to be met.

Causes of Failure of Sheet-Pile Dams.—Failures in sheet-pile dams have generally occurred in those of the spillway or overflow type, which are commonly called diversion weirs. The seepage water which finds its way under a line of sheet-piling tends to buoy up the particles of sand on the down-stream side, while the surface of the river bed is frequently subjected to a variable degree of scouring action from the current of water which spills over the crest. These two forces combined move the finer particles of sand first, then, as the interstices in the sand gradually increase in size, the upward flow of seepage water increases, both in force and volume, and the destruction of the dam is in progress.

It is customary with many engineers to make the length of sheet-piling such that the depth of penetration below the bed of the stream will be approximately equal to the head of water to be impounded by the dam or weir, and it is likely that many of the failures of such structures, which have been attributed to "unknown" causes, were due to the inadequacy of the cut-off wall under the structure—more often, however, on account of carelessness in driving and faulty alignment than from insufficient depth of penetration.

Length of Sheet-Piling Required.—Capillary attraction and surface friction, in the interstices between the grains, are forces of such magnitude in retarding the velocity of seepage flow through sand, or other pervious materials, that, with reasonable depth of penetration of a curtain-wall of sheet-piling, the direct pressure which may result from any reasonable head of water impounded by the dam cannot produce a velocity of seepage flow sufficiently rapid to move the fine particles of sand without the added force of the overflow current flowing on the surface of the river bed; therefore, in designing the dam or diversion weir, adequate provision must be made to counteract the effects of both of these forces.

Assuming that the force of capillary attraction effectively resists the pressure head of the impounded water, in its natural function of

producing an acceleration of seepage flow, then the body of sand on the down-stream side may be considered as a mass which is held together by the force of capillary attraction, and that the pressure due to the head of water on the up-stream side passes under the line of sheet-piling and acts on a wedge-shaped mass of sand on the down-stream side.

The hydrostatic pressure at the bottom of the sheet-piling is exerted equally in all directions, but, we are here concerned only with the force which acts in an upward direction. The tendency of this force to buoy up the mass of sand is resisted by the excess weight, or specific gravity, of the solid particles which compose the body of sand, after deducting the percentage of voids which are taken up by the water.

Let h = Maximum head of water on up-stream side,
d = Depth of penetration for sheet-piling,
s = Specific gravity of material penetrated,
x = Proportion of solids.

For average sand, $s = 2.65$ and $x = 1 - 0.40 = 0.6$.

Then $d = \frac{h}{s \times x} = \frac{h}{2.65 \times 0.6} = \frac{h}{1.59} = 0.629\,h$, which is the theoretic penetration.

This equation provides for the depth of penetration at which the excess hydrostatic pressure on the up-stream side is theoretically counterbalanced by the specific gravity of the depth of material on the down-stream side, in sand of such size and porosity that the combined effect of friction and capillary attraction is a maximum.

A length of sheet-piling which provides for a depth of penetration equal to the maximum head at flood-stage, therefore, involves a factor of safety of only 1.59 (with a tight wall of sheet-piling in perfect alignment) applied to the theoretic minimum. This is manifestly inadequate for such variable and uncertain conditions as are likely to be encountered.

Where the workmanship, alignment, and depth of penetration of a curtain-wall of sheet-piling are such as to ensure that there will be no marked "leakage" flow through openings, or places of variable density, silt and matter carried in suspension by the current of the stream will settle in the quiet pool of water impounded by the dam, and gradually seal up the interstices in the sand, until the bed of the stream becomes practically impervious. This has been thoroughly demonstrated in

irrigation canals constructed in sandy soil, and by the fact that the graded sands used in the filter beds of many of our larger cities require frequent renewal, or removal, for the purpose of washing out such accumulations, in order that even the customary slow rate of percolation may be maintained.

No material, other than solid bed-rock, offers a more secure support than sand which is properly confined; therefore, to make it serve as a safe foundation for a permanent dam is entirely a problem of making adequate provision against the only disturbing element—a current of flowing water.

For dams which are to be constructed on a sand foundation, and embody the special features of construction recommended in a subsequent part of this paper, the writer suggests the following empirical rules:

$d = 2.5\ h$......For heads up to 8 ft.....................(I)

$d = 2\ h$........For heads up to 15 ft...................(II)

$d = 1.6\ h$......As the minimum, in any case..........(III)

Equation (III) uses the proportion of excess weight, due to the specific gravity of the sand, as a coefficient instead of as a divisor. It provides a factor of safety of 2.5 under normal conditions of low head, and, as the velocity of seepage flow through sand probably varies as some function of the square root of the head, the factor of safety is correspondingly increased for higher heads.

Special Features of Construction.—The horizontal distance through which seepage water must flow, after passing under a line of sheet-piling, before it reaches an outlet at the surface, is an important element in retarding its force to such extent that there will be no erosion, or movement, of the sand composing the surface of the river bed. It is especially important below spillways, or dams of the overfall type, where the velocity of current from the overflow of the dam may already be such that erosion of the finer particles of sand takes place; therefore, the added impetus of upward percolation would produce a dangerous condition of erosion and the inevitable destruction of the dam.

An apron below a dam serves, not only to resist the shock or abrasion of falling water, but also to seal the bed of the stream, so that the seepage water which finds its way under the curtain-wall of sheet-piling, must flow diagonally upward beyond the limits of the

apron before issuing. The width of such apron, therefore, is an important element in fixing the practical factor of safety of the structure, and should not be overlooked.

Aprons for spillways, diversion weirs, and dams of the overfall type, should be submerged, so as to form a water-cushion to absorb the shocks of falling water, ice, and débris. Such water-cushion should have a depth equal to one-quarter, and preferably one-third, of the total height of overfall, and a down-stream width of not less than $1\frac{1}{2}$ times the total height of overfall, with the outer edge forming an apron, sloping upward, and extending down stream for a distance of not less than $1\frac{1}{2}$ to 2 times the length of sheet-piling used in the main curtain-wall, ending with, and supported on, a secondary line of sheet-piling of from one-third to one-half the depth of penetration of the curtain-wall.

A special feature of construction, which provides for considerable economy in the quantity of concrete required for a wide apron, and, at the same time, provides a greater element of safety against the force of seepage flow than a solid concrete apron of the dimensions stated above, is illustrated in Fig. 1.

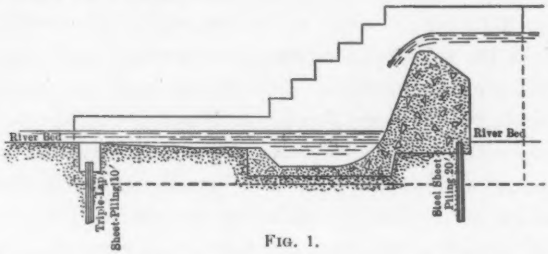

FIG. 1.

The submerged apron should be constructed in the form of a trough, with a depth of not less than one-fourth, and a down-stream width of $1\frac{1}{2}$ times the height of the overfall, and the outer edge should be finished off at, or slightly below, the elevation of the bed of the stream. At a point down stream, a distance of about twice the length of the main sheet-piling used (or 4 or 5 times the head), there should be driven a second cut-off wall of tight sheet-piling with a depth of penetration of about one-half that of the main curtain-wall, parallel to the line of the dam, to support a substantial concrete curb which should be finished off at the elevation of the bed of the stream.

The writer wishes to emphasize the statement that: One of the

most serious problems involved in the design and construction of dams for rivers with beds of shifting sands is that of providing adequate protection against dangerous erosion of the river bed for some distance below the dam, where the erosive action of the current is dangerously magnified by reason of the upward force of seepage flow, under the variable hydrostatic pressure communicated from the head of water impounded on the upper side of the dam.

In the design suggested in Fig. 1, the open space between the edge of the trough forming the water-cushion and the concrete curb supported by the secondary line of sheet-piling provides an open space for the escape of the seepage flow which finds its way under the curtain-wall, while the concrete curb prevents the erosion and removal of the sand, which, in this confined section of the river bed, is subject to the combined forces of upward and horizontal currents.

The approximate general dimensions suggested apply to low dams of the diversion-weir class, such as are used to divert water from wide and shallow streams for purposes of irrigation and power; and the importance of careful and conscientious workmanship on every detail of construction in sand is so self-evident that special mention seems to be unnecessary.

Every dam or diversion weir should be designed in accordance with the practical demands of the exact local conditions which prevail. This involves modifications of any set of general rules, especially for higher heads, that is, for dams which, in sandy rivers, will raise the level of the water more than 12 or 15 ft. For low dams there is a variety of shapes, or cross-sections, from which to choose; but, for higher heads, economy in the matter of the materials involved narrows down the choice in the direction of the curved or ogee form of cross-section.

Under the general rules previously stated, a dam which is designed to raise the level of the water 50 ft., in a river having a bed of sand of unlimited depth, would require a curtain-wall of sheet-piling having a depth of penetration of not less than 80 ft. below the bed of the stream, as the minimum. A dam of this height would probably be designed with an ogee form of curved cross-section. The impracticability of driving a line of wooden sheet-piling to such depth in sand and gravel, and the expense of providing a curtain-wall of steel sheet-piling of such lengths, would suggest a careful analysis of modifica-

tions, for the possibility of eliminating a portion of such construction without impairing its stability and permanence.

Ample assurance of safety could be attained by using a line of steel sheet-piling of such length as to provide a depth of penetration of 50 ft. under the up-stream side of the dam, and a secondary line of sheet-piling with a depth of penetration of about 15 ft. under the extreme toe, which should be curved up so as to form a trough at a depth of about 5 ft. below the bed of the river, for a water-cushion. Then, at a point about 200 or 250 ft. down stream, construct a smaller dam, about 15 or 16 ft. in height, with a line of main sheet-piling having a depth of penetration of 30 ft., and embodying all other features of design in accordance with the special features outlined in this paper, to back up the water against the main dam for a proper depth of water-cushion. The seepage flow, which would probably find its way under the main dam in considerable quantities, would issue from the section of confined river bed between the two dams, where the depth and length of the water-cushion thus provided prevents the removal of any sand, because, even during periods of storm flow, the violent abrasive force of the flood passing over the dam would be effectually broken by the extra depth of water-cushion which is provided by the concrete trough at the toe of the dam.

In calculating the proportions of a dam which is to be founded on sand, there is involved the consideration of the hydrostatic pressure against the under side of the dam.

The calculations for the cross-section of a masonry dam founded on bed-rock are based on the assumption of an absolutely water-tight contact between the footings and the bed-rock, therefore the entire cross-sectional weight of the dam may be used in calculating the moment of resistance to overturning.

In a dam of such proportions that the resultant of forces passes close to the toe, the effect of water working its way under the footings at points of defective contact, or through defective joints in the lower courses of masonry, might produce a change in the direction of the resultant such that it would pass outside of the toe. That section of the dam would then be in unstable equilibrium, and, unless restrained by other forces, would overturn.

In considering the forces which may act on a dam resting on sand, saturation of the foundation, with upward hydrostatic pressure,

is an accepted fact, and, therefore, must be considered as a definite element in the calculations.

As stated previously, the combined effect of friction and capillary attraction offers such marked resistance to the flow of water through sand that the velocity which results from any head, or inclination of surface, is infinitesimal, in comparison with the velocity which would result under similar conditions of head in a body of free water flowing in an open channel, or through a pipe. The transmission of hydrostatic pressure through a similar medium is similarly resisted, to a slight degree, as will be shown.

The removal of the soil above a stratum of dense clay, which in turn overlies a stratum of water-bearing sand or gravel under Artesian pressure, will disclose a moist degree of saturation of the clay such that water will accumulate in small pockets, or sumps, which may be bored or dug to shallow depths of 1 or 2 ft. into the clay. The seepage flow which is thus indicated is absorbed by the soil above by capillary attraction, and is evaporated from the surface; however, no hydrostatic pressure, of measurable magnitude, would be exerted against the footing of any structure resting on such a stratum of clay.

Hydrostatic pressure is directly proportional to the head of water, and time is not a factor in determining its magnitude, hence the coefficient of friction becomes zero. Clay contains a relatively large proportion of voids of infinitesimal size, and, while a high head of water pressing against the under side of a stratum of clay which is confined between two porous strata would undoubtedly transmit hydrostatic pressure of measurable magnitude through such stratum of clay, it is evident that, under moderate heads, the force of capillary attraction (in interstices of such infinitesimal size that they would almost warrant an assumption of molecular friction), is of sufficient magnitude to counteract and practically nullify the effect of hydrostatic pressure.

It may be concluded, therefore, that the force with which hydrostatic pressure is transmitted through alluvial soils is modified by a coefficient which varies as the square, cube, or perhaps "*n*th" power of the effective size of the interstices, or, inversely as such function of the effective size of the grains.

For the practical purposes of this paper, the probable existence of

such a coefficient may be disregarded, because, for the interstices of the sizes which are ordinarily found in sand of 40% porosity, its probable value becomes so small as to be negligible.

The hydrostatic pressure against the under side of any dam of the type and general dimensions illustrated in Figs. 1 and 2 is governed by the head of water on the down-stream side of the dam, because the pressure which is transmitted under the curtain-wall of sheet-piling, from the head of water above the dam, is afforded lateral outlet, therefore its intensity is limited to the depth of resistance, or head, opposing such outlet on the down-stream side.

The hydrostatic uplift per square foot of surface against the base of a dam of the type and general proportions recommended in this paper, therefore, will vary between 25 and 40% of that which would result from the head of water on the up-stream side.

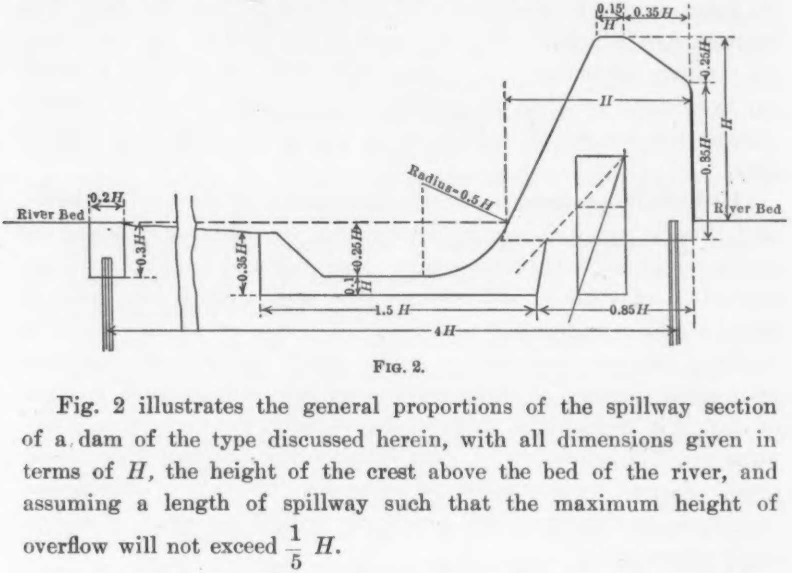

FIG. 2.

Fig. 2 illustrates the general proportions of the spillway section of a dam of the type discussed herein, with all dimensions given in terms of H, the height of the crest above the bed of the river, and assuming a length of spillway such that the maximum height of overflow will not exceed $\frac{1}{5}$ H.

The resultant, which is obtained when the calculated weight of the section of concrete is reduced by the total amount of hydrostatic uplift due to a down-stream head $= 0.3\ H$, intersects the base practically in the center, as is shown by the solid line. This would indicate that the design may be modified so as to involve less concrete, but, the quantity to be thus saved, in the total cost of a low dam,

will not outweigh the desirability of such a factor of stability in a structure of this class.

The dotted line indicates the resultant which would be obtained if the weight of the concrete were reduced by an amount equal to the full hydrostatic uplift which would result from a head equal to the maximum up-stream head = 1.3 H. This resultant intersects the base outside of the middle third, but well within the toe.

The sloping portion of the top of the dam, on the up-stream side of the crest, is desirable to facilitate the passage of ice and floating débris over the crest, and to reduce the moment of ice thrust during periods of extreme cold when the entire flow of the stream may be diverted for power purposes.

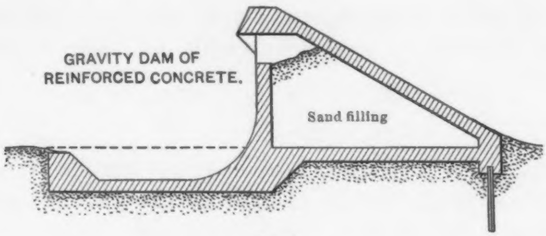

Fig. 3.

Any increase in the width of the base, with the additional material on the up-stream side of the dam, so as to lengthen the inclined portion, affects the moment of stability of the structure, on the principle of the well-known "gravity" type of dam.

Dams to be constructed on alluvial foundations should be designed with the maximum width of base which is consistent with economy; therefore, the gravity type of dam, constructed of reinforced concrete, is especially adapted to the requirements.

In low wooden structures of the gravity type, there is danger that they may float during a period of storm-flow of such depth that the gravity head, on which such dams depend for their stability, may be destroyed. By the use of reinforced concrete, however, this source of danger may be eliminated.

Figs. 3 and 4 indicate two variations of design for gravity dams of reinforced concrete which permit of removing the forms and filling the compartments with sand or other heavy material, in order to give additional weight to the structures, or, if suitable air vents

are provided, the compartments may be allowed to become filled with water.

From the simple inclined platform to the large hollow structures of the Ambursen type, the gravity dam of reinforced concrete offers

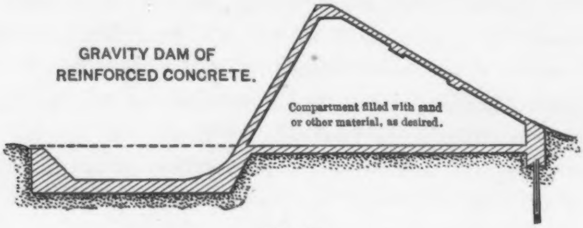

FIG. 4.

a sufficiently wide range of variation to meet the requirements of almost any locality, or any individuality of taste in engineering design.

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PAPERS AND DISCUSSIONS

A DISCUSSION OF EXPERIMENTS ON RETAINING WALLS, AND OF PRESSURES ON TUNNELS.*

By William Cain, M. Am. Soc. C. E.

The most extended experiments relating to retaining walls are those pertaining to retaining walls proper and the more elaborate ones on small rotating retaining boards. The results referring to the former agree fairly well with a rational theory, especially when the walls are several feet in height; but with the latter, many discrepancies occur, for which, hitherto, no explanation has been offered.

It will be the main object of this paper to show that the results of these experiments on small retaining boards can be harmonized with theory by including the influence of cohesion, which is neglected in deducing practical formulas. It will be found that the influence of cohesion is marked, because of the small size of the boards. This information should prove of value to future experimenters, for it will be shown that, as the height of the board or wall increases, the influence of cohesion becomes less and less, so that (for the usual dry sand filling) for heights, say, from 5 to 10 ft., it can be neglected altogether.

The result of the investigation will then be to give to the practical constructor more confidence in the theory of the sliding prism, which serves as the basis of the methods to follow.

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

As, in the course of this investigation, certain well-known constructions for ascertaining the pressure of any granular material against retaining walls will be needed, it is well to group them here. The various figures are supposed to represent sections at right angles to the inner faces of the walls with their backings of granular material. In the surcharged wall, Fig. 1, produce the inner face of the wall to meet the surface of the surcharge at F. It is desired to find the thrust against the plane, AF, for 1 lin. ft. of the wall. Draw AD through A, the foot of the wall, making the angle of repose, ϕ, of the earth with the horizontal and meeting the upper surface at D. Since any possible prism of rupture, as $FACR$, in tending to move downward, develops friction against both surfaces, AC and AF, the earth thrust on the wall will make an angle, ϕ', with the normal to AF, where ϕ' is the angle of friction of the earth on the wall. As the earth settles more than the wall, this friction will always be exerted. Again, as the wall, from its elasticity and that of the foundation, will tend to move over at the top on account of the earth thrust, the earth, with its frictional grip on the wall, will tend to prevent this, so that the friction is exerted downward in either case, and the direction of the earth thrust, E, on AF is as given in Fig. 1.

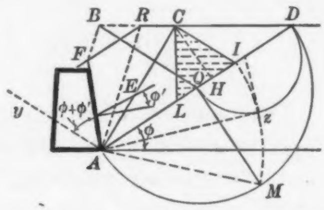

FIG. 1.

However, if $\phi' > \phi$, a thin slice of earth will move with the wall, and the rubbing will be that of earth on earth, so that ϕ' in this case must be replaced by ϕ. This rule will apply in all cases that follow, without further remark, wherever ϕ' is mentioned.

Now draw Ay, making the angle, $\phi + \phi'$, with AF, as shown; then draw FB parallel to AR, to the intersection, B, with DR produced. From B a parallel to Ay is constructed, meeting AD at O.

Since theory gives the relation: $AI = \sqrt{AD \cdot AO}$, two constructions follow, by geometry, for locating the point, I. By the first, a semicircle is described on AD as a diameter; at the point, O, a perpendicular is erected to AD, meeting the semicircle in M; then AI is laid off equal to the chord, AM. By the second construction, a semicircle is described on $O\,D$ as a diameter, a tangent to it, Az, from A is drawn, limited by the perpendicular radius, and finally AI is laid off equal to Az.

The point, I, having been thus found by either construction, draw IC parallel to Ay to the intersection, C, with BD. AC is the plane of rupture. On laying off $IL = IC$, and dropping the perpendicular, CH, from C on AD, the earth pressure, E, on AF is given by $\frac{1}{2}\ CI\ .\ CH\ .\ e$, where e is the weight of a cubic unit of the earth; otherwise, the value of E is given by e times the area of the shaded triangle, ICL. If the dimensions are in feet, and e is in pounds per cubic foot, the thrust, E, will be given in pounds.

In Figs. 2 and 3, the retaining boards, AB, are vertical, and BO is drawn, making the angle, $\phi + \phi'$, with the vertical, AB. The upper surface of the earth is BD, and the constructions for locating I and C are the same as for Fig. 1. AC, in all the figures, represents the plane of rupture.* In all cases, the earth thrust found as above is supposed to make the angle, ϕ' (as shown) with the normal to the inner wall surface.

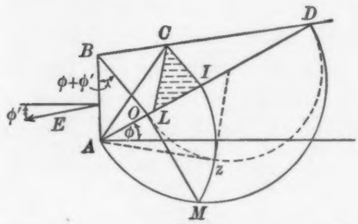

FIG. 2.

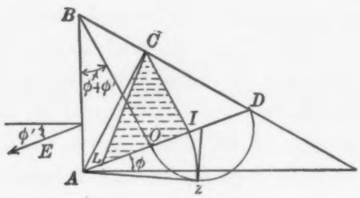

FIG. 3.

In the Rankine theory, pertaining, say, to Fig. 2, the earth thrust on a vertical plane, AB, is always taken as acting parallel to the top slope. This is true for the pressure on a vertical plane in the interior of a mass of earth of indefinite extent, but it is not true generally for the pressure against a retaining wall. Thus, when BD, Fig. 2, is horizontal, Rankine's thrust on AB would be taken as horizontal, which entirely ignores the friction of the earth on the wall. The two theories agree when $\phi' = \phi$ and BD slopes at the angle of repose, in which case, as BC is parallel to AI, there is no intersection, D. It is a limiting case in which, to compute the thrust, CIL can be laid off from any point in BD, on drawing CI parallel to BO, etc. As BD approaches the natural slope, the point, D, recedes indefinitely to

* The writer refers to his "Retaining Walls," Van Nostrand's Science Series, No. 3, for the demonstrations pertaining to the above constructions, and to the derivations of formulas.

the right, and it is seen that the plane of rupture, *AC*, approaches indefinitely the line, *AD*, or the natural slope. This limiting case, on account of the excessive thrust corresponding, will be examined more carefully in the sequel.

If h = the height of the wall, *AB*, in feet, and e = the weight of a cubic foot of earth, in pounds, then when $\phi' = \phi$, and the surface *BD*, Fig. 2, slopes at the angle of repose, the earth thrust, in pounds, is given by the equation:

$$E = \frac{1}{2} e h^2 \cos. \phi \qquad \qquad (1)$$

If, however, ϕ' is not equal to ϕ, then E is directed at the angle, ϕ', to the normal to the wall, and the thrust is:

$$E = \frac{1}{2} e h^2 \frac{\cos.^2 \phi}{\cos. \phi'} \qquad \qquad (2)$$

The foregoing constructions, and the corresponding equations, are all derived from the theory of the sliding prism. The wedge, *BAC*, Figs. 2 and 3, is treated as an invariable solid, tending to slide down the two faces, *AB* and *AC*, at once, thus developing the full friction that can be exerted on these faces. In the case of actual rotation of the board, *AB*, it is found by experiment that each particle of earth in the prism, *BAC*, moves parallel to *AC*, each layer parallel to *AC* moving over the layer just beneath it.

A similar motion is observed if the board, *AB*, is moved horizontally to the left. However, in the first case (of rotation) the particles at *A* do not move at all, whereas in the second (of sliding motion) the particles about *A* move, rubbing over the floor, which thus resists the motion by friction. A thrust, thus recorded by springs or other device, in the case where the wall moves horizontally, would give an undervaluation at the lower part of *AB* and consequently the computed center of pressure on *AB* would be too high. On that account, only the experiments on rotating boards will be considered in this paper.

The theory of the sliding wedge, however, is justified, because no motion of either kind is actually supposed. The wedge, *BAC*, is supposed to be just on the point of motion, it being in equilibrium under the action of its weight, the normal components of the reactions of the wall, and the plane, *AC*, and all the friction that can be exerted along *AB* and *AC*. These forces remain the same, whatever incipient motion

is supposed. The hypothesis of a plane surface of rupture, however, is not exactly realized, experiment showing that the earth breaks along a slightly curved surface convex to the moving mass. For the sake of simplicity, the theory neglects the cohesion acting, not only along *AC*, but possibly to a small extent along *AB*. This additional force will be included in certain investigations to be given later.

These preliminary observations having been disposed of, the results of certain experiments on retaining walls at the limit of stability will now be given.

Figs. 4, 5, 6, and 7 refer to vertical rectangular walls backed by sand, except in the case of Fig. 5, where the filling was macadam screenings. The surface of the filling was horizontal in each case.

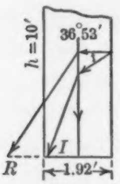

FIG. 4.

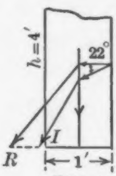

FIG. 5.

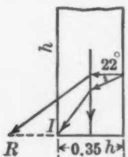

FIG. 6.

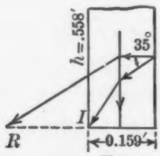

FIG. 7.

To give briefly in detail the quantities pertaining to each wall, the following symbols will be used:

h = height of wall, in feet;
t = thickness of wall, in feet;
e = weight of filling, per cubic foot;
w = weight of wall, per cubic foot;
ϕ = angle of repose of filling;
ϕ' = angle of friction of filling on wall;
q = distance from center of pressure on the plane of the base to the outer toe, divided by t, using the theory that rationally includes the whole of the wall friction;
q_0 = the same, according to the Rankine theory.

q and q_0 are positive when the resultant on the base strikes within the base, otherwise they are negative.

Fig. 4 represents Lieutenant Hope's wall of bricks laid in wet sand: $h = 10$, $t = 1.92$, $e = 95.5$, $w = 100$, $\phi = \phi' = 36° 53'$. It was 20 ft. long, and was backed by earth level with its top. $q = + 0.04$, $q_0 = - 0.58$. The overhang, at the moment of failure, was probably 4 in. Including this, $q = - 0.02$.

Fig. 5 shows Baker's wall of pitch-pine blocks, backed by macadam screenings, the level surface of which was 0.25 ft. below the top of the wall; $h = 4$, $t = 1$, $e = 101$, $w = 46$, $\phi = 39° 48'$, $\phi' = 22°$, the assumed angle of friction of timber on stone, $q = -0.06$, $q_0 = -0.55$.

Trautwine's experimental wall is shown in Fig. 6. Only the ratio of base to height, 0.35, was given by the author, but J. C. Trautwine, Jr., Assoc. Am. Soc. C. E., assures the writer that the walls were probably 6 in. in height, though certain notes refer to walls varying from about 4 to 9 in. $e = 89$, $w = 28$, $\phi = 33° 42'$, ϕ' (assumed) $= 22°$, $q = -0.03$, $q_0 = -0.75$.

The wall of Curie, Fig. 7, was of wood coated on the back by sand, so that $\phi = \phi' = 35°$. Also, $h = 0.558$ ft., $t = 0.159$ ft., $\frac{e}{w} = 3.29$, $q = 0.02$, $q_0 = -1.32$.

These walls were all at the limit of stability, and the first two are of appreciable height, 10 ft. and 4 ft., respectively.

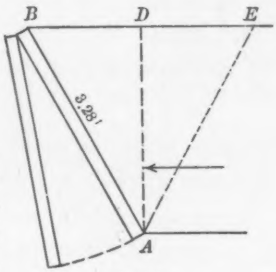

FIG. 8.

The figures show that the theory, including the whole of the wall friction, agrees fairly well with experiment, but that the Rankine theory does not thus agree. In both theories, the thrust, E, is supposed to act at one-third of the height from the base of the wall to the surface of the filling; but, in the Rankine theory, this thrust is assumed to act horizontally, whereas, in the other theory, it is supposed to act in a direction making the angle, ϕ', below the normal to the wall.

On combining the thrusts with the weight of the wall, as usual, the resultant strikes the base produced, at R in the first case (Rankine theory), but at I in the second case. Figs. 4 to 7 present a striking object lesson as to the inaccuracy of the Rankine method of treating experimental retaining walls.

In the next experiments, however, referring to a retaining structure consisting of two boards, hinged at the top, Fig. 8, and backed by sand level at the top, the Rankine theory is applicable when the board, AB, is placed either at or below the plane of rupture, on the left of AD. The thrust on AD is then assumed to act horizontally, at $\frac{1}{3}$ AD above A, and is combined with the weight of the sand, DAB, to find the

resultant on the board. If the board is at the plane of rupture, this resultant will make the angle ϕ below the normal to AB; hence, if one assumes a less thrust on AD, especially if inclined downward, the new resultant on AB will make an angle greater than ϕ with the normal to AB, which is inconsistent with stability.* The same reasoning applies when AB lies below the plane of rupture.†

The retaining board, 1 m. square, was coated with sand, so that $\phi = \phi' = 35°$ for damp sand. Hence, for a horizontal thrust on AD, the plane of rupture (which bisects the angle between the vertical and the natural slope) makes an angle of 27° 30′ with the vertical. The board, AB, was set at this angle to the vertical, sand was filled in level with the top, and it was found that the structure was at the limit of stability when $AC = 0.45$ m. In the meantime, however, the sand had dried out, so that ϕ was 33° 30′; hence, strictly, the construction of Fig. 1 (for earth level with top of wall) applies; but, as the results can only differ inappreciably, the thrust on AD, acting horizontally, was computed for $\phi = 33°$ 30′ and combined with the weight of sand, BAD, and the weight of structure, both acting through their centers of gravity, to find the resultant on the base, AC. It was found to cut it 0.11 of its width from the outer toe, C; therefore $q = 0.11$.

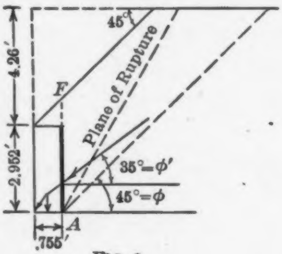

FIG. 9.

In the next experiment, the angle, BAD, was 55°, $AC = 0.548$ m. and $\phi = 33°$ 30′. Pursuing the same method, it is found that $q = 0.02$, or the resultant on the base passes practically through C. The third experiment was on a smaller retaining board. Here $AB = 0.2$ m., $BAD = 55°$, $\phi = \phi' = 35°$, and $q = 0.06$.

In Fig. 9 is shown a surcharged wall of Curie's, just at the limit of stability, having $h = 2.952$ ft., $t = 0.755$ ft. and the level upper surface of the surcharge being 4.26 ft. above the top of the wall. The surcharge extended over the wall at the angle, $\phi = 45°$, corresponding to damp sand. Experiment gave $\phi' = 35°$. The wall was of brick in Portland cement. The ratio, $\frac{e}{w} = 0.62$. It was found, using the

* A full discussion may be found in the writer's "Retaining Walls."

† The experiments pertaining to Figs. 7, 8, and 9 are due to Curie. See Curie's "Poussée des Terres" and "Trois Notes," Gauthier-Villars, Paris. They are of especial interest in that they were undertaken to attempt to overthrow the theory advocated above.

construction of Fig. 1, that taking the thrust, E, as acting 1.24 ft. above the base, or at one-third of the height of the surface, AF, that $q = + 0.03$; and further, that if E acts 1.303 ft. above the base, the resultant on the base passes exactly through the outer toe of the wall.

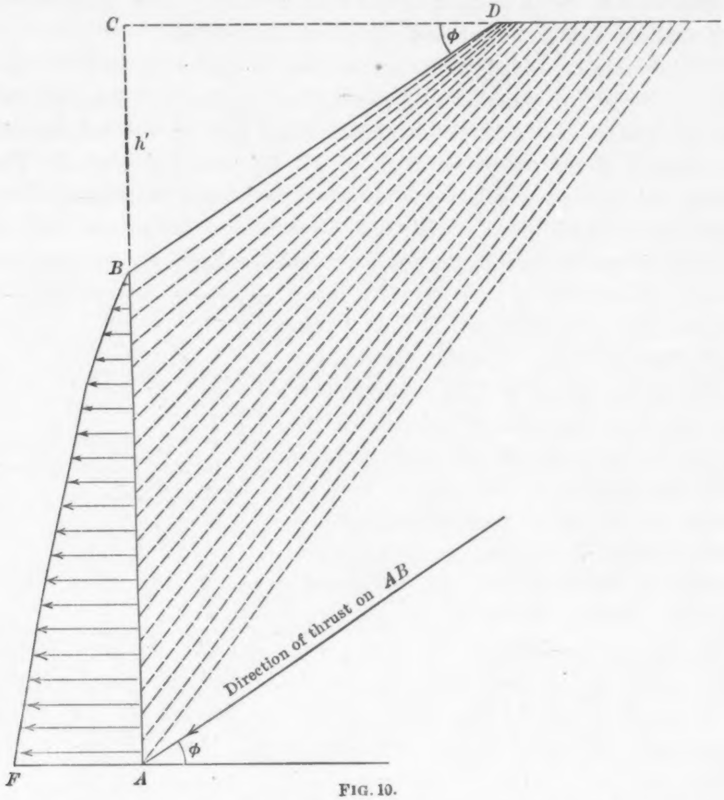

FIG. 10.

As the true position of the center of pressure on a surcharged wall has never been ascertained, as far as the writer knows, he has made a number of constructions, after the method illustrated in Fig. 1, in order to find it.

In place of making the construction for the special case above, it was thought that the results would be more generally useful if the natural slope was taken with a base of 3 and a rise of 2, and $\phi' = \phi$, therefore $\phi = \phi' = 33° \ 41'$. The wall, AB, Fig. 10, was taken vertical and 20 ft. high. The surcharge sloped from B at the angle

of repose to a point, *D,* 10 ft. above *B,* from which point the surface of the earth was horizontal. The face of the wall, *AB,* was divided into twenty equal parts, 1 ft. each; and, by the construction of Fig. 1, the thrusts (inclined at the angle, ϕ, below the normal to the wall) were found for the successive heights of wall of 1, 2, 3, ... 20, ft., respectively, taking the weight of 1 cu. ft. of earth equal to unity. The successive planes of rupture are shown by the dotted lines in Fig. 10. On the original scale (2 ft. to 1 in.), the upper plane of rupture (for a height of wall = 1 ft.) was found to pass slightly to the right of *D.*

On subtracting successive thrusts, the thrusts on each foot of wall were obtained. These were plotted as horizontal ordinates at the center of each foot division of the wall, and the "peaks" were slightly rounded off, as shown on the figure. Since, with all care, mistakes amounting to 1% of the total thrusts can easily be made, it was proper to adjust the results in this manner to give the most probable unit pressures on the successive divisions of the wall. The centers of pressure, for heights of the wall varying from 5 to 20 ft., were easily obtained by taking moments about some convenient point; the results are given in Table 1.

Call h the height of wall, measured from B downward, and $h' = BC$, the height of surcharge above the top of the wall; also, let c = the ratio of the distance from the foot of the wall considered to the center of pressure, to the height of the wall. The values of c, for various ratios, $\frac{h'}{h}$, are given in Table 1.

TABLE 1.

$\frac{h'}{h}$	c	$\frac{h'}{h}$	c	$\frac{h'}{h}$	c	$\frac{h'}{h}$	c
∞	0.333	1.50	0.356	1.00	0.364	......	......
......	......	1.25	0.360	0.75	0.364	0.00	0.333
2.00	0.353	1.11	0.362	0.50	0.364		

It is seen, as $\frac{h'}{h}$ diminishes, that c increases, until for $\frac{h'}{h} = 1$, the maximum value for c, 0.364, is attained and remains the same up to $\frac{h'}{h} = 0.50$, after which it probably diminishes, because, for $\frac{h'}{h} = 0$, $c = \frac{1}{3}$.

When some other flatter slope is given to BD, doubtless these values of c will be altered, but, for the case supposed, they should prove serviceable in practice.

Although the earth thrusts on successive portions of AB are really inclined at $\phi°$ below the normal to AB, they are laid off here at right angles to it, so that the area, ABF, is equal to the total thrust on AB. If the unit pressures varied as the ordinates to the straight line, BF, as for a uniformly sloping earth surface, then, as is well known, $c = \frac{1}{3}$. The area to the left of BF gives the excess thrust which causes c to exceed $\frac{1}{3}$.

Making use of the results of the table as approximately applicable in the foregoing example (Fig. 9), and taking the center of pressure on AF as $0.364\,AF$ above the base, the resultant there is found to pass 0.02 outside of the base, therefore $q = -0.02$. This experiment on a surcharged wall, of the kind shown, is particularly valuable as being the only one of which any account has been given, as far as the writer knows.

Recurring once more to Fig. 10, it may be recalled that some authors have assumed the unit pressures on AB to vary as the ordinates to a trapezoid, so that the unit pressure at B was not zero (as it should be), but an amount assumed somewhat arbitrarily. In particular, Scheffler derived in this way $c = 0.4$ as an upper practical limit, and used it in making tables for use in practice.

A remark must now be added (relative to all the experimental walls previously mentioned, except Trautwine's), that the friction of the backing on the sides of the box in which the sand was contained has been uniformly neglected. Where the wall is long, this can have little influence, but where the length is not much greater than the height, as in the experiments, this side friction becomes appreciable.

Darwin, as well as Leygue, endeavored to estimate the amount the full thrust (with no side friction) was reduced, by experimenting with sand behind a retaining board, or wall, enclosed in a box as usual, when a partition board was placed perpendicular to the wall and centrally in the mass, and comparing results with those found when the partition board was omitted. Leygue thus found, for walls having a length of twice the height, that the true or full thrust was diminished about 5% from the side friction, for level-topped earth, and as much as 15% for the surface sloping at the angle of repose.

If this is true, then the experimental walls just considered would have to be thicker to withstand the true actual thrust; or to put it another way, for the given thickness, the theoretical thrust, including the side friction, would have to be made (as a rough average) about 5% less for the level-topped earth and (roughly) 15% less for the earth sloping at the angle of repose. From the figures it is seen that this will modify the results but slightly, not enough to alter the general conclusion that the theory advocated (including the wall friction) is practically sustained by the experiments, and that the Rankine theory is not thus sustained.

Trautwine's wall consisted of a central portion of uniform height, from which it tapered to the ends, the upper surface being at the angle of repose for the tapered ends. In this case no side friction was developed. The results agree in a general way with the others.

In the many experiments on high grain bins, the enormous influence of the friction of the grain against the vertical walls or sides of the bin has been observed. In fact, the greater part of the weight of grain, even when running out, is sustained by the walls through this side friction. This furnishes another argument for including wall friction in retaining-wall design.

In connection with this subject, it may be observed that many experiments, made to determine the actual lateral pressure of sand or its internal friction angle, are inconclusive, because an unknown part of the vertical pressure applied to the sand in the vertical cylinder or box was sustained by the sides of the cylinder or box. The ratio of lateral to vertical pressures, or the friction angle, cannot be precisely found until the proportion of the load sustained by the sides of the containing vessel has been ascertained experimentally. The writer is of the opinion that the best experiments to aid in the design of retaining walls are those relating to the rotation of retaining walls or boards. The few given herein are the best recorded, though some of them were on models which were too small. In fact, for the small models of Leygue and others, the effect of cohesion is so pronounced that some of the results are very misleading.

As the experiments by Leygue* were very extensive, and evidently made with great care, they will be considered carefully in what follows.

* All the experiments of Leygue referred to in what follows may be found in *Annales des Ponts et Chaussées*, November, 1885.

As preliminary to the discussion, however, it is well to give the essentials of Leygue's experimental proof that cohesion and friction exist at the same time. A box without a bottom, about 4 in. square in cross-section and 4 in. high, was made into a little carriage by the addition of four wheels. The latter ran on the sides of a trough filled with sand which the bottom of the box nearly touched. The box was partly filled with sand, and the trough and box were then inclined at the angle at which motion of the box just began, the sand in the box resting on the sand in the trough, developing friction or cohesion or both, just before motion began. Only friction was exerted after motion began. The solution involves the theory of the inclined plane, but, to explain the principles of the method, it will suffice to suppose the trough and the sand in it to be horizontal, and that the bottomless box filled with sand is just on the point of moving, due to a horizontal force applied to it. The weight of the box and a part of the weight of the sand in it held up by the friction of the sides, is directly supported by the wheels resting on the sides of the trough; so that only a fraction of the weight, W, of the sand in the box is supported directly by the sand in the trough. Call this amount $n\,W$. Then, for equilibrium, calling P the horizontal force, less the resistance of the carriage wheels, we have,

$$P = A\,k + n\,W\,f,$$

where A = the area of the cross-section of the box,
k = the force of cohesion per unit of area,
and f = the coefficient of internal friction of the sand.*

The value of n was found by weighing: For the dry sand it varied from 0.79 to 0.65, for heights of the sand in the box varying from 1.2 to 3.5 in. For the damp sand and fresh earth (slightly moistened and slightly rammed) which can stand with a vertical face for the height of the box, the filling was loosened by many blows on the box, and n was taken equal to 1.

Three suppositions were made: (1) that both cohesion and friction acted at the same time before motion; (2) that friction alone acted ($k = 0$); (3) that cohesion alone acted ($f = 0$).

* We can suppose, here, the horizontal force to be the pull of a cord extending horizontally from the box and passing over a fixed pulley, and that at the free end of the cord a weight is applied. The friction of the pulley and carriage wheels could be found experimentally and allowed for, so that some fraction of this weight would equal P.

The results for various heights of sand in the box are given in Table 2.

TABLE 2.

	(1)		(2)	(3)
	k	f	f	k
Dry sand	7	0.70	0.80 to 0.96	26 to 56
Wet sand	40	0.85	1.20 " 1.90	73 " 133
Very wet sand	31	1.70	2.00 " 2.40	107 " 226
Fresh earth	90	1.63	2.60 " 4.40	150 " 242

The values of k are given in kilogrammes per square meter. It is seen that suppositions (2) and (3) give discordant results, whereas (1), for each kind of filling, gave identical values of k and of f for various heights; hence it may fairly be concluded that, before motion, cohesion and friction both acted at the same time. As to the high values found for k, for the coherent fresh earth, Leygue states that Collin found, by an independent method, for clayey earth and clay of little consistency, $k = 113$ and $k = 193$, respectively. As a further verification of the values of k and f given in (1), it is found that, on using them in the formula for computing the height at which the wet sand or earth will stand vertically, the results agree with experiments.

The values of k, in pounds per square foot, given in Column (1), with the values of ϕ corresponding to the f given, are as follows:

$$\begin{aligned} \text{Dry sand, } k &= 1.47, \ \phi = 35^\circ \\ \text{Wet sand, } k &= 8.28, \ \phi = 40^\circ\ 22' \\ \text{Very wet sand, } k &= 6.36, \ \phi = 59^\circ\ 30' \\ \text{Damp fresh earth, } k &= 18.45, \ \phi = 58^\circ\ 28' \end{aligned}$$

It is possible that the method used by Leygue may prove of service to experimenters in obtaining more accurately than hitherto the coefficient of internal friction. Increasing pressures could be obtained by adding weights on top of the sand in the box; but, unless the total weight sustained by friction along the sides of the box is determined carefully for each weight used, the results can have but little value. Further, for coherent earths, the method of Leygue is open to objections.

Admitting the hypothesis that cohesion and friction act at the same time, a general graphical method* will now be given to find the thrust against the inner face, *AB,* of a retaining wall or board, Fig. 11, caused by the earth, $AB\ b_6\ H$, tending to slide down some plane of rupture, $A\ b_1$, $A\ b_2$, ..., the resistance along this plane being due both to friction and cohesion.

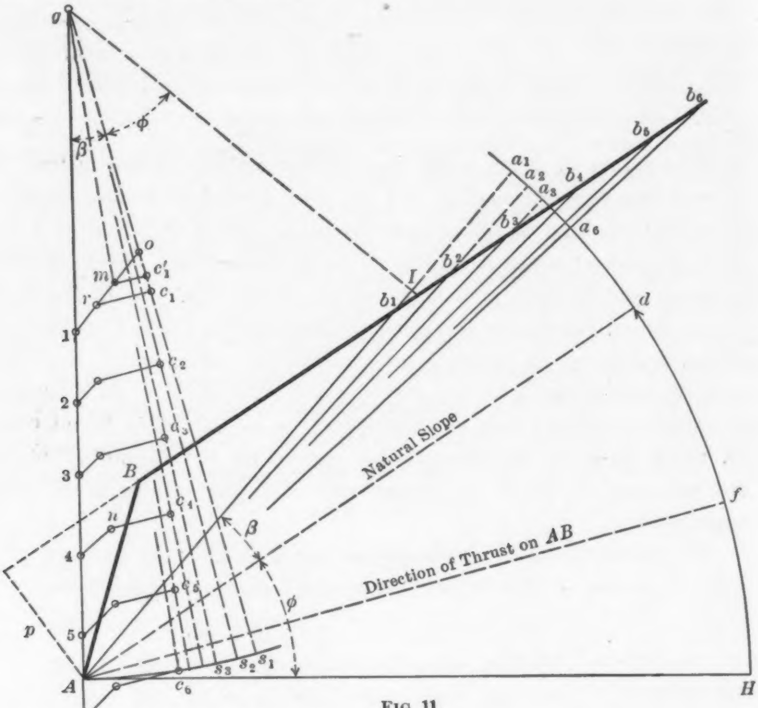

FIG. 11.

Suppose $A\ b_1$ to be the plane of rupture, and call the weight of the prism of rupture, $AB\ b_1$, for a thickness of one unit, W. The weight of the prism causes the tendency to slide along the planes, $A\ b_1$ and *AB.* This tendency is resisted by the reactions of the wall, *AB,* and the plane, $A\ b_1$. The reaction of the wall consists of the normal component, E_1, acting to the right, and the friction resistance, E_1 tan. ϕ', acting up. The resultant of these two forces, E, which is equal and opposed to the earth thrust on *AB,* thus makes the angle, ϕ',

* This method is an extension of that given by Professor H. T. Eddy in his treatment of earth thrust, in "Researches in Graphical Statics."

with the normal to the wall. Its direction is given by $A f$ in Fig. 11. The reaction of $A\ b_1$ is made up of the cohesion, C, acting up along $A\ b_1$, the normal component, N, acting up, and the friction, N tan. ϕ, acting up along $A\ b_1$. The two forces, N and N tan. ϕ, when combined, give a resultant, S, making an angle ϕ with the normal, $g\ I$, to $A\ b_1$. Hence, if the angle, $I\ g\ s_1 = \phi$, then $g\ s_1$ gives the direction of the resultant, S.

The prism, $AB\ b_1\ A$, is thus in equilibrium under its own weight, W, the cohesive force, C, acting up along $A\ b_1$, the reaction, E, of AB, acting to the right, and the force, S, acting up. On drawing, to the scale of force, g 1 vertical and equal to W; then 1 r parallel to $A\ b_1$ and equal to C; then $r\ c_1$ parallel to $A f$ to the intersection with $g\ s_1$, the sides of the closed polygon, g 1 $r\ c_1\ g$, in order, will represent the four forces, W, C, E, and S, in equilibrium.

A similar investigation pertains to any other supposed prism of rupture. To find the true one, a number of trial planes of rupture, $A\ b_1 . A\ b_2, \ldots,$ are assumed, and each is treated in turn as a true one (though there can be only one true one). As seen above, the resultant of the normal reaction and friction on any trial plane of rupture must be inclined below the normal to the plane at the angle, ϕ. To lay off the directions of these resultants, from any convenient point, g, say, in the vertical through A, as a center, describe an arc, $A\ s_1$, with a convenient radius, $g\ A$. With the same radius and A as a center, describe the arc, $H\ f\ d\ a_1$, cutting the trial planes (produced if necessary) at $a_1, a_2, \ldots, a_6$. Let d be the point where the line of natural slope from A cuts the arc, $H\ a_1$. On laying off the chords, $A\ s_1, A\ s_2, \ldots,$ equal to the chords, $d\ a_1, d\ a_2, \ldots,$ respectively, it will follow that $g\ s_1, g\ s_2, \ldots,$ will make angles, ϕ, below the normals to the planes, $A\ a_1, A\ a_2, \ldots,$ respectively. To prove this, take any trial plane, as $A\ b_1$, which makes the angle, β, with $A\ d$, and drop a perpendicular, $g\ I$, from g on $A\ b_1$ (produced if necessary); then, because the sides of the angles are perpendicular, $A\ g\ I = H\ A\ a_1 = \phi + \beta$, and if $A\ g\ s_1 = \beta$, it follows that $I\ g\ s_1 = \phi$, as was to be proved. Hence the chord, $A\ s_1 =$ the chord, $d\ a_1$, $A\ s_2 = da_2$, etc., as stated.

The weights, in pounds, of the prisms, $A\ B\ b_1, A\ B\ b_2, \ldots,$ are, $\frac{1}{2}\ ep . B\ b_1, \frac{1}{2}\ ep . B\ b_2, \ldots,$ respectively, where p is the length of the perpendicular from A upon $B\ b_1$ (produced if necessary), the foot

being the unit of length and e being the weight, in pounds, of 1 cu. ft. of earth. The prisms are supposed to be 1 ft. in length perpendicular to the plane of the paper.

These weights are now laid off to the scale of force, vertically downward from g, to points 1, 2, 3, ..., and from these points, lines are drawn parallel to $A\,b_1$, $A\,b_2$, $A\,b_3$, ..., respectively, of lengths equal to $k\,.\,A\,b_1$, $k\,.\,A\,b_2$, ..., to represent the forces of cohesion, acting upward along $A\,b_1$, $A\,b_2$, ..., where $k =$ the force of cohesion, in pounds per square foot. From the extremities of these lines, lines are drawn parallel to the direction of the earth thrust on AB (inclined at the angle, ϕ', below the normal to AB), to the intersections, c_1, c_2, c_3, ..., with $g\,s_1$, $g\,s_2$, $g\,s_3$, ..., respectively. With dividers, it is found, for this figure, that $n\,c_4$ is the longest of these lines; whence $n\,c_4$, to the scale of force, measures the earth thrust against AB, in pounds. This follows, because, for any less thrust, since n is fixed, when $n\,c_4$ becomes less, c_4 falls to the left of the first position, and the new $g\,c_4$, representing the thrust on $A\,b_4$, due to the normal reaction on it and friction only, will make a greater angle than ϕ to the normal to $A\,b_4$, which is inconsistent with the laws of stability of a granular mass. In fact, if N is the normal component of the thrust on the plane, $A\,b_4$, N tan. ϕ is all the friction that can be exerted on it. The resultant of N and N tan. ϕ thus makes an angle, ϕ, with N, and this angle cannot be exceeded. The true thrust on the wall, AB, is thus the greatest of the trial thrusts, $n\,c_4$. The prism of rupture, $A\,b_4\,B$, is in equilibrium under the four forces represented by the sides of the closed polygon, $g\,4\,n\,c_4\,g$; $g\,4$, representing its weight; $4\,n$, the cohesion acting along $A\,b_4$; $n\,c_4$, the reaction of AB (opposed and equal to the earth thrust); and $c_4\,g$ the reaction of the plane, $A\,b_4$, due to the normal component and friction on it only. The full reaction of the plane can be found by combining the forces, given in magnitude and direction by $4\,n$ and $c_4\,g$, but it is not needed.

It is to be noted that $n\,c_4$ is the least thrust for which equilibrium is possible. The other trial thrusts should now be lengthened to equal $n\,c_4$, since this is the true thrust or reaction of the wall, $A\,B$. All the new points, c_1, c_2, c_3, c_5, c_6, will now lie to the right of the old points; hence the new $g\,c_1$, $g\,c_2$, $g\,c_3$, $g\,c_5$, $g\,c_6$, will all make less angles than ϕ with the normals to the planes, $A\,b_1$, etc.; hence stability everywhere in the earth mass is assured.

The solution represented by Fig. 11 is general, and applies whether AB is inclined to the right or left of the vertical through A or coincides with it, and whether the earth surface $B\,b_6$ is horizontal or inclined above or below the horizontal. It can likewise be easily adapted to the case shown in Fig. 1.* The construction of Fig. 11 has been used in evaluating the thrust and determining (approximately) the plane of rupture in the experiments (recorded below) of Leygue on retaining boards, AB, that could be rotated about A, and thus placed at any inclination to the vertical. In all the experiments, the vertical height of AB was 0.656 ft.; the length of the board was 1.3 ft. The value of the moment of the earth thrust about A was found by use of suitable apparatus, corresponding to dry sand with a natural slope of 3 base to 2 rise, or $\phi = 33° \ 41'$, $\phi' = \phi$, and $c = 89$ lb. per cu. ft. By use of the partition board mentioned previously, the side friction of the sand on the glass sides of the box containing it was estimated, and the moments corrected, so as to give the true moment when there is no side friction. The notation used to express results is partly given in Fig. 12, for the general case where,

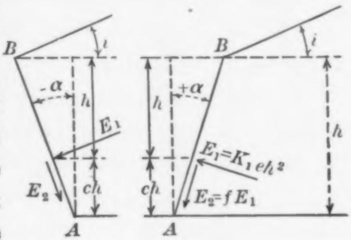

FIG. 12.

h = vertical height of the board, AB;

ch = vertical height to center of pressure on AB;

α = angle AB makes with the vertical, counted as positive or negative according as AB lies to the right or left of the vertical through A;

i = inclination of surface of earth to the horizontal;

$E_1 = K_1 \ e \ h^2$ = normal component of thrust on AB, acting $c\,h$ feet vertically above A;

$E_2 = f\ E_1$ = tan. $\phi'\ E_1$ = component of thrust acting in the direction from B to A, f = tan. ϕ';

γ = angle plane of rupture makes with horizontal.

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* To attain the greatest accuracy, in constructions like that shown in Fig. 11, the scale should be as large as possible; the arcs of circles, especially, must be drawn with a large radius, and the points, s_1, s_2, etc., determined with care. The angle, $H\ A\ f$, can be computed and laid off by aid of a table of chords. The construction in this figure corresponds to a vertical height of $A\ B$ = 0.656 ft., $k = 1$, $e = 80$. The value of the component, E_1, perpendicular to $A\ B$, is now to be found, by drawing lines from n and c_4, perpendicular and parallel to $A\ B$, to intersection, and measuring the component to scale. For $k = 2$, it is found that $A\ b_1$ is the plane of rupture. The line, $m\ c_1'$, through the new c_1 (c_1'), representing the thrust, is very small; but it can be easily magnified by laying off the polygon, $g\ 1\ m\ c_1'$, to a scale two or three times as large, and thus the thrust can be found as accurately as before.

The resultant, E, of E_1 and E_2, evidently makes the angle, ϕ', with the normal to AB. The moment of this resultant about $A = M = E_1$ $c\ h$ sec. $\alpha = c\ K_1$ sec. $\alpha\ .\ e\ h^3 = m\ e\ h^3$, if we put $m = c\,K_1$ sec. α. From the last formula, it is seen that m is the moment of the thrust about A, for $e = h = 1$; also from $E_1 = K_1\ e\ h^2$, it follows that K_1 is the normal component of the thrust for $e = h = 1$.

When cohesion is included, c is not exactly $\frac{1}{3}$, but it is very slightly less for $k = 1$ or 2. It will be assumed at $\frac{1}{3}$, and, from the values of m given by Leygue, K_1 will be derived from the formula above,

$$K_1 = 3\ m \text{ cos. } \alpha.$$

Thus, for the case represented by Fig. 11, tan. $\alpha = \frac{1}{3}$, therefore $\alpha = 18° 26'$, cos. $\alpha = 0.949$; tan. $i = \frac{2}{3}$. Experiment gave $m = 0.032$, therefore $K_1 = 3 \times 0.032 \times 0.949 = 0.091$.

Neglecting cohesion, theory gives $K_1 = 0.182$, or twice the amount given by experiment. If, however, the construction of Fig. 11 is made for the actual height of the retaining board, $h = 0.656$ ft., $e = 80$, $k = 1$ (cohesion, in pounds per square foot), we find $E_1 = 2.75$. On substituting this in the formula, $E_1 = K_1\ e\ h^2$, we have,

$$2.75 = 80\ (0.656)^2\ K_1, \text{ therefore } K_1 = 0.080.$$

By a comparison of the values, it is evident that, if the cohesion was assumed at a little less than 1 lb. per sq. ft., the theoretical and experimental values could be made to agree exactly. The case just examined exhibits the most pronounced difference between the ordinary theory (corresponding to $k = 0$) and experiment, of any shown in Table 3. Further, it will be observed, that, for an assumed cohesion of about 1 lb. per sq. ft., the theoretical and experimental values for all the cases given by Leygue very nearly agree.

The value, $e = 80$, in place of Leygue's, $e = 89$, was used, which would alter the results somewhat, but not the general conclusions. The construction of Fig. 11 will give E and its normal component, E_1, with practical accuracy, but it is not readily adaptable in finding the plane of rupture. In most of the drawings a small scale was used, in order to limit the drawing to a sheet of writing paper, hence, on both accounts, γ cannot be counted on to nearer than 1° or 2°, except for $k = 0$, when γ was found by computation, or by the construction of Fig. 1.

TABLE 3.

Tan. a.	Tan. i.	Cohesion, k, in pounds per square foot.	Angle of rupture with the horizontal, γ.		Coefficient K_1 of the normal component of the thrust $E_1 = K_1 e h^2$.	
			Theory.	Experiment.	Theory.	Experiment.
$+\frac{1}{3}$	0	0	50° 12′	51° 30′	0.060	0.043
		1	51°		0.042	
		2	52° 30′		0.026	
$+\frac{1}{3}$	$\frac{2}{3}$	0	33° 41′	47°	0.182	0.091
		1	44°		0.084	
		2	49°		0.043	
0	0	0	56° 36′	56° 30′	0.111	0.090
		1	57°		0 093	
		2	58°		0.077	
		3	58° 30′		0 062	
0	$\frac{1}{2}$	0	47° 30′	51°	0.178	0.141
		1	50°		0.148	
		2	53°		0.121	
		3	55°		0 098	
0	$\frac{2}{3}$	0	33° 41′	49°	0.345	0.195
		1	44°		0.205	
		2	46° 30′		0.150	
		3	50°		0.111	
$-\frac{1}{3}$	0	0	60° 21′	61°	0.185	0.179
		1	63°		0.171	
		2	63°		0.155	
$-\frac{1}{3}$	$\frac{2}{3}$	0	33° 41′	57°	0.660	0.387
		1	57°		0.267	
		2	50°		0.236	

The results in Table 3 are remarkable, and explain quite satisfactorily how Leygue, Darwin, and others found, by experiments on small models, results differing so much from the ordinary theory, where cohesion is neglected.

It should be remarked that the values of γ, given in Table 3 under "Experiment," are not exactly those given by Leygue in his tables, but are the averages obtained from the two sets of drawings given by him in the plates, and represent the inclinations of the chords of the really curved surfaces of rupture. His experiments with the spring apparatus

are not considered, as the results are open to doubt, because the prism of rupture, in descending, could not slide down freely, but as it advanced would rub over the floor, thus lessening the thrust there considerably.

From Table 3, the results given by experiment are seen to differ widely from the ordinary theory in which $k = 0$.

The discrepancies are largely, or almost entirely, due to the very small models used, as will be evident from the following considerations: Suppose the height, h, of the wall, AB, to be 10 times the height given in Fig. 11, or 6.56 ft.; then, as the areas of triangles such as $AB\,b_1$, etc., vary as the squares of the heights, but the lengths of sides, as $A\,b_1$, etc., vary only as the first power of the heights, the weights of the successive trial prisms of rupture will be 10^2 or 100 times as great as before, whereas the corresponding cohesive forces, acting along the planes, $A\,b_1$, etc., will be only 10 times the first values. Hence, if we use a scale of force $\frac{1}{100}$ of the former scale, the weights of the prisms, $AB\,b_1$, $AB\,b_2$, etc., will be represented, as before, by g 1, g 2, etc., but the lines representing the cohesive forces will be only $\frac{1}{10}$ of the former lengths. Thus the new 4 n, Fig. 11, will be laid off from 4 only $\frac{1}{10}$ of the length shown in the figure.

The relative decrease in the lines representing cohesive forces will be still more marked for a wall $20 \times 0.656 = 13.12$ ft. high, the weights of prisms being 400 times as great, but the cohesive forces only 20 times as great as before. It is evident from this reasoning that, for $k = 1$, the cohesive forces are practically negligible for walls, say, 10 ft. high, especially if the earth surface is level. In fact, a little examination of the original drawings showed, for walls about 6 ft. high, that the earth thrust, neglecting cohesion, was only from 1 to 5% in excess over that for $k = 1$. The smaller percentages referring to tan. $\alpha = -\frac{1}{3}$, or tan. $\alpha = 0$, while the larger percentages referred to tan. $\alpha = +\frac{1}{3}$, for earth surface horizontal or sloping at the angle of repose.

Such results should be of great service to future experimenters as proving two things: (1) that dry sand, with as small a coefficient of cohesion as possible, should be used (perhaps grain would be a more suitable material), and (2) that no experimental wall should be less than from 6 to 10 ft. high.

Even if the wall is, say, 6 ft. high, if damp clayey earth is used

as the filling, with a coefficient of adhesion, $k = 10$, then all the diagrams of forces, as in Fig. 11, will be the same as before, or similar figures, and the discrepancies noted in Table 3, will be as pronounced as ever. All the experiments on retaining boards, except some of Curie's, have been with very small models, and the results have brought the common theory under suspicion, if not into disrepute.

The writer hopes that the foregoing investigation and results may be instrumental in establishing more confidence in the theory, and in showing when cohesive forces may be practically neglected and when they must be included.

As an illustration, the results for a vertical wall 10 ft. high are presented in Table 4; taking $\phi = 33° 41'$ and $e = 80$. In the first wall, the surface of the earth was horizontal, in the second wall its slope was 1 rise to 2 base.

TABLE 4.

Tan. α.	Tan. i.	k.	γ.	K_1.
0	0	0	56°36′	0.111
		1	56°	0.110
		5	57°	0.101
		10	57°	0.096
0	$\frac{1}{2}$	0	47°30′	0.178
		1	49°	0.176
		5	49°	0.165
		10	49°	0.155

In Table 4 the results for $k = 0$ and $k = 1$ are practically the same, but K_1 for $k = 10$ is 13% less than for $k = 0$. If the value, $k = 18.5$, for fresh earth slightly damp and lightly rammed, given by Leygue above, is even approximately correct, it is seen that, for such a filling, the effect of cohesion must be included to get results at all agreeable with experience or experiment.

Recurring to the experimental retaining walls proper, Figs. 4 to 9, it is evident from the foregoing, that cohesion will affect the results inappreciably, except perhaps in the case of Figs. 6 and 7, where the height was about 0.6 ft. Assuming $k = 1$, it seems to be probable, from the results of Table 4, that the thrust should be decreased in the ratio of 93 : 111. Effecting the construction for the new thrust, it is found that the point, I, falls within the base, 0.03 of its width for Fig. 6 (Trautwine's wall), and 0.02 of its width for Fig. 7 (Curie's wall).

The theory advocated is thus practically sustained by all the experiments given above, either on retaining boards or retaining walls proper, when a coefficient of cohesion of about $k = 1$ for dry sand is used.

The method of evaluating the thrust, given in Fig. 11, is as valid when $k = 0$, or cohesion is neglected, as in the ordinary theory. The lines parallel to the thrust are now drawn directly from 1, 2, ..., to the intersection with $g\,c_1$, $g\,c_2$, ..., and the greatest one is taken for the true thrust. Although the writer expressly disclaims any great accuracy in the values of K_1 in Table 4, on account of the small scale of the drawings, nevertheless, the results by the construction for $k = 0$ and tan. $i = 0$ or $\frac{1}{2}$, were found to differ from computed values only 2, 3, 0, and 1 % for the different cases, which should give confidence in the general conclusions, at least.

The diagram, Fig. 11, with a slight modification, can be utilized to find the coefficient of cohesion, k, at which the bank of earth will stand without a retaining board. Thus, let each line, as $4\,n$, representing the cohesive force acting along its proper plane, be extended to meet the corresponding $g\,c$; any such line measured to the scale of force and then divided by the length of the plane along which it acts, will give the cohesive force, in pounds per square foot, corresponding to no thrust on AB, for the particular plane considered. The greatest of these values is evidently the value of k for which the filling will stand without a retaining board. The work can be much abbreviated by using a well-known principle, that the plane along which the unit cohesion is greatest (the plane of rupture) bisects the angle, $BA\,d$, between the surface, AB, and the line of natural slope. Suppose $A\,b_1$ to be this plane, then we have only to extend $1\,m$ to meet $g\,c_1$, at 0, measure 10 to the scale of force, and divide by the length of $A\,b_1$, to the scale of distance, to find the coefficient desired. By either method it was found that a cohesive force of 7 lb. per sq. ft. was required to sustain a mass of earth with a vertical face, $AB = 0.656$ ft. high, when $B\,b_6$ was horizontal.

It was stated, in connection with Equations (1) and (2), referring to the thrust on a vertical wall of height, h, with the earth surface sloping at the angle of repose, that this particular case would be discussed later. To show the influence of cohesion, the planes of rupture for such a wall, 2.4 ft. high, for various values of k (in pounds per square

foot), are given in Fig. 13. The values of K_1 (for $e = 80$ and $\phi = \phi' = 33° 41'$) are as follows:

$k = 0 \ldots\ldots\ldots K_1 = 0.345$ $k = 5 \ldots\ldots\ldots K_1 = 0.183$
$k = 2 \ldots\ldots\ldots K_1 = 0.237$ $k = 10 \ldots\ldots\ldots K_1 = 0.120$
$k = 3 \ldots\ldots\ldots K_1 = 0.215$ $k = 15 \ldots\ldots\ldots K_1 = 0.075$

The first value was found by computation, the others by construction. As is well known, the theoretical plane of rupture approaches indefinitely the natural slope as k approaches zero. For appreciable cohesion (and there is always some cohesion) the plane of rupture lies above the natural slope, with very materially decreasing normal components to the thrust as k increases. As the height of wall increases, the influence of cohesion diminishes. Thus, as shown above, for a wall 5 times 2.4 ft., or 12 ft. high, the weights of the prisms, ABC, etc., are 25 times the former values, but the cohesive forces, which vary directly as AC, etc., are only 5 times the first values. Hence, if the former values of k are multiplied by 5, the new diagram of forces, Fig. 11, will be similar to the old one. Thus, for the wall 12 ft. high, the plane of rupture and the value of K_1, for $k = 10$, correspond to the old values for $k = 2$, for $k = 15$, to the old values for $k = 3$. For fresh earth filling, slightly packed, it is possible that the values, $k = 10$, $k = 15$, may be reached, with a material reduction in K_1 from the values given by Equations (1) and (2). As the height of the wall increases, say to 25 or 50 ft., the influence of cohesion, in diminishing the thrust, becomes very small, and it is better to ignore it altogether. In fact, as we know very little, and that imperfectly, of the coefficients of cohesion, it is perhaps safer, at present, to use Equations (1) and (2) in all cases. It is very evident, though, that for most cases in practice, the formulas give a very appreciable excess over the true thrust, and that the true plane of rupture never coincides with the natural slope.

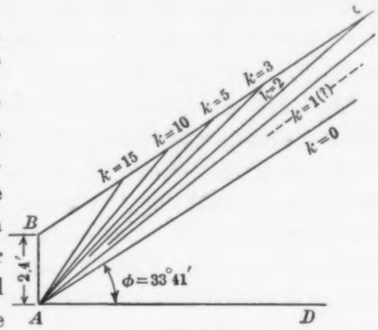

FIG. 13.

From all that precedes, it is seen that the results of experiments on small models in the past have proved to be very misleading, and that experiments on large models are desirable, and can alone give confidence. Leygue has made such experiments on retaining boards, from 1 to 2 m. (3.28 to 6.56 ft.) in height, simply to determine the surface of rupture. This is really the essential thing, for, as soon as the prism of rupture is known, the thrust is easily found. In a general way, the results agree with theory when the cohesion is neglected, though the curved surfaces of rupture were very irregular, particularly for the stone filling. The first two experiments were made with both dry and damp sand as a filling; the next six, with stones varying from 1.5 to 20 in. in diameter. In another series of five experiments, sand was used. In all the foregoing experiments, the surface of the material was horizontal. In three additional experiments, the walls were surcharged with sand as a filling. In one experiment, the wall was 6.56 ft. high and the surcharge was 3.28 ft.; in another experiment, the wall was 3.28 ft. high, and the sand, sloping from its top at the angle of repose, as in the former case, extended to 3.28 ft. above the wall, where the surface was horizontal.

Applying the construction of Fig. 1, it was found that the plane of rupture passed, say, 2° above that given by experiment in the first case and about 3° below in the second. It will be evident from the construction of Fig. 11, omitting cohesion, that trial planes of rupture differing by 2 or 3° from the true one, give nearly the same thrust. Taking the average, these experiments on large models, tend, in a general way, to sustain the theory.

In the late Sir Benjamin Baker's paper, "The Actual Lateral Pressure of Earthwork,"* two experiments by Lieutenant Hope and one by Colonel Michon, on counterforted walls, are given. Although such walls do not admit of precise computation, on account of the unknown weight of earth carried by the counterforts, through friction caused by the thrust of the earth in a direction perpendicular to the counterforts, still the computation was made, as the conclusions are interesting. Therefore, the first vertical wall of Lieutenant Hope was examined, especially as Mr. Baker, using the Rankine theory, found, for this wall, the greatest divergence between the actual and the Rankine thrust, of any retaining wall examined.

* *Minutes of Proceedings*, Inst. C. E., Vol. LXV, p. 140; reprinted in Van Nostrand's Science Series.

At the moment of failure, the wall was 12 ft. 10 in. high, the thickness of the panel was 18 in., and the counterforts were 10 ft. from center to center, projecting 27 in. from the wall, or 3 ft. 9 in. from the face, as inferred from the next example. As it is stated that the wall had the same volume as the 10-ft. wall previously examined in this paper (Fig. 4), the counterforts must have been 2 ft. thick. Assuming these dimensions, and using the values given; tan. $\theta = \frac{3}{4}$, or $\theta = 36° \ 52'$ (say $= \phi'$), $e = 95.5$ lb. = weight of a cubic foot of earth, and $w = 100$ lb. = weight of a cubic foot of masonry, we first compute $E_1 = 1\,496$ lb., the normal component of the earth thrust on a length of 1 ft. of wall. The normal thrust on the panel is thus $8\ E_1$ and on the counterfort $2\ E_1$. The friction (acting vertically downward) caused by this thrust is $8\ E_1$ tan. θ on the panel and $2\ E_1$ tan. θ on the counterfort. The moment of these forces about the outer toe of the wall, totals 39 800 ft-lb. The resisting moment of 10 ft. in length of combined panel and counterfort, about the outer toe, assuming the wall to be vertical, is 29 800 ft-lb. If, to the latter, we add the moment of 17% of the weight of earth between the counterforts, supposed to be held up by the sides of the latter, the total moment exactly equals the first. However, at the moment of failure by overturning, the panels had bulged 4½ in. and the overhang at the top was 7½ in. Taking the moment of stability of the wall at 26 000 ft-lb. (Mr. Baker's figure), it is found that, for equilibrium, 24% of the weight of earth between the counterforts must be carried by them. When the earth was 8 ft. high, a heavy rain was recorded, so that, doubtless, some appreciable cohesion was exerted, though necessarily omitted in the computation.

The experimental wall of Colonel Michon was 40 ft. high, with very deep counterforts, only 5 ft. from center to center. The very heavy and wet filling between the counterforts, being treated as a part of the wall, a construction (made on the printed drawing) shows that the resultant of earth thrust and weight of wall passes through the outer toe. Doubtless the cohesion factor in this wall was large. In the paper mentioned, the details as to General Burgoyne's experimental walls are given. There were four of these walls, each 20 ft. long, 20 ft. high, and with a mean thickness of 3 ft. 4 in. Two of the walls were perfectly stable, as in fact theory indicates for all four walls if they were monolithic. The other two walls fell, one bursting out

at 5 ft. 6 in. from the base, and the other (a vertical wall), breaking across, as it were, at about one-fourth of its height. As these walls consisted of rough granite blocks laid dry, it is highly probable that the breaks were due to sliding, owing to the imperfect construction; besides, "the filling was of loose earth filled in at random without ramming or other precautions during a very wet winter."

From a consideration of all the observations and experiments (some of them unintentional), Mr. Baker concludes that the theoretical thrust is often double the actual lateral pressure. He used the old theory, which neglects both cohesion and wall friction. If he had included them, the resulting theory would not have been so deficient "in the most vital elements existent in fact" as he charges against the "textbook" theory.

However, the writer must be clearly understood as not recommending that cohesive forces be considered in designing a retaining wall backed by a granular material, such as fresh earth, sand, gravel, or ballast. It has been the main object of this paper to show that, although cohesive forces must be included in interpreting properly the results on small models and many retaining walls, yet, for walls more than 6 or 10 ft. in height, backed with dry fresh material, not consolidated, the cohesive forces can be practically neglected in design. Hence, experimenters are strongly advised to leave small models severely alone and confine their experiments to walls from 6 to 10 ft. high, backed by a truly granular material, such as dry sand, coal, grain, gravel, or ballast, where the cohesive forces will not affect the results materially. Further, it is evident that walls of brick in wet sand, or walls of granite blocks, etc., laid dry, are very imperfect walls. The overhang, just before falling, is large, and the base is often imperfect. For precise measurements, a light but strong timber wall on a firm foundation, seems to be best; and the triangular frame of Fig. 8 seems to meet the required conditions very well, especially if the framing is an open one, with a retaining board only on one leg. The base thus becomes wider and the overhang less, than with any rectangular wall.

When the design of a wall to sustain the pressure of consolidated earth is in question, even if a perfect mathematical theory existed, it would still prove of little or no practical value, because the coefficients of friction and cohesion are unknown. The coefficient of friction at the surface can be easily found, but it is a difficult matter to find the

coefficient of cohesion, which doubtless varies greatly throughout the mass.

Mr. W. Airy, in his discussion of Mr. Baker's paper, states that he found the tensile strength of a block of ordinary brick clay to be 168 and of a certain shaley clay 800 lb. per sq. ft., the coefficients of friction for the two materials being 1.15 and 0.36, respectively. Cohesive resistance is more analogous to shear, but such figures indicate the wide variations to be expected, particularly in k, the coefficient of cohesion. If this coefficient is to be guessed at, in order to substitute it in the supposed perfect formula, then it is plainly better to guess at the thickness of the wall in the first instance.

As an illustration, consider the well-known equation:*

$$h = \frac{4\,k}{e}\,\frac{1 + \sin.\,\phi}{\cos.\,\phi} = \frac{4\,k}{e}\left(f + \sqrt{1 + f^2}\right)$$

which gives the height, h, of vertical trench that will stand without any sheeting.

In this equation,

k = the cohesive or shearing resistance, in pounds per square foot;

e = the weight of a cubic foot of earth, in pounds;

$f = \tan.\,\phi$;

ϕ = the inclination of the natural slope; and

h = the height of the unsupported bank, in feet.

Thus, if $f = 0.7$, whence $\phi = 35°$, the equation reduces to

$$h = 7.68\,\frac{k}{e}$$

For, $e = 100$, $k = 100$, $h = 7.68$ ft.,
$e = 100$, $k = 200$, $h = 15.36$ ft.,
$e = 100$, $k = 300$, $h = 23.04$ ft.

As certain trenches with vertical sides have been observed to stand unsupported for heights of 15 or even 25 ft., the equation would seem to indicate that cohesive or shearing resistances of about 200 to 300 lb. per sq. ft. were required to cause equilibrium. If friction is not supposed to be exerted, then $f = 0$ and $h = 4\,\frac{k}{e}$; and, for the same unsupported heights, the cohesion would be about doubled. Evi-

* In reference to this equation, see Appendix.

dently, if cohesion, which (to judge from Mr. Airy's experiments) may vary from one to several hundred pounds per square foot, has to be guessed at in order to determine h, it is plainly better to guess at h at once.

The foregoing equation cannot be regarded as giving very accurate results, mainly because a plane surface of rupture is assumed, whereas, from both theory and observation, this surface is known to be very much curved; besides, the cohesion and friction along the ends of the break have been neglected. However, the hypothesis of a plane surface of rupture, the ends being supposed to be included, gives a greater value to h than the true one, whereas, neglecting the influence of the ends, it tends in the other direction; so that the equation may not err so greatly.

In the discussion of the paper* by J. C. Meem, M. Am. Soc. C. E., E. G. Haines, M. Am. Soc. C. E., states that where breaks occur in the sides of an unsupported trench, the solid rupture often approximates to a quarter-sphere, surmounted by a half-cylinder of the same height, the radii of the sphere and cylinder being equal. In Fig. 14, let $AIBE$ represent the quarter-sphere, ABG the half-cylinder, and $EACDB$ the face of the trench. According to the observations of Mr. Haines, when the part, $ACDB$, of the side of the trench is supported by sheeting and bracing, it sometimes happens that a part of the quarter-sphere, $AIBE$, breaks out, so that the semi-cylinder above would descend but for the bracing, the thrust of which, it is supposed, induces arch action in the earth.

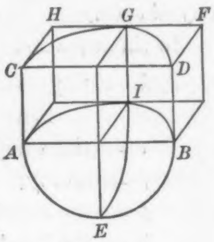

FIG. 14.

This is possible; but, if so, as the sheeting is not supposed to be carried to the bottom of the trench, there can be no vertical component in its reaction, and the thrust, B, of the braces and sheeting, acting on $ACDB$, must be horizontal; further, the earth cannot act as a series of independent arches devoid of frictional resistance between them, but must act as a whole.

Another way of explaining the phenomena is to suppose the horizontal thrust of the braces, B, on the exposed face, $ABCD$, to cause

* "The Bracing of Trenches and Tunnels, With Practical Formulas for Earth Pressures." *Transactions*, Am. Soc. C. E., Vol. LX, p. 1. A number of important facts brought out in this paper are of vital importance to constructors.

friction at the back of the break of sufficient intensity to prevent the semi-cylinder from descending, just as a book can be held against a vertical wall by a horizontal push.

To illustrate the principle, it will suffice to replace the semi-cylinder by the circumscribing parallelopiped, AF, and suppose it to be held up by the friction on the back face, with possibly cohesion acting on the three interior vertical faces. Thus, let $AC = CH = a$, and $AB = 2a$; then the friction on the back face is Bf, the cohesion on the three faces is $(2a^2 + 2a^2)k$, and the weight of earth, AF, equals $2a^3e$. Hence, as friction and cohesion always act opposite to the incipient motion, or vertically upward in this case,

$$Bf + 4a^2k = 2a^3e.$$

Evidently, the value of B, derived from this equation, gives an extreme upper limit, which is doubtless never attained, as there is nearly always some support from the earth which has not broken out below the level of ABI.

Where the sheeting and bracing are of sufficient size, are tightly keyed up, and extend to the bottom of the trench, or where the bank is supported by a retaining wall, the earth near the bottom cannot break out, and the equation is not valid.

However, if, from any cause, such as insufficient sheeting, the break has taken place over even a part of $AIBE$, the mass, AF, above will tend to tip over at the top, giving the greatest pressure on the top braces. This appears to explain the phenomena observed by Mr. Meem and others in connection with some trenches.

With regard to tunnel linings, as is well known, the vertical pressure on the top is generally small, the great mass of earth vertically over the tunnel being largely held up by the friction of the earth (caused by the earth thrust) on its vertical sides, exactly as in the case of tall bins, where most of the weight of the grain is held up by the sides of the bin, the theory being very similar in the two cases. In consolidated earth, cohesion assists very materially in this action.

It might be inferred, from the facts of observation, that consolidated earth acts as a solid, though, of course, it differs from a solid in this: that its physical constants (cohesion, friction, etc.) vary enormously with the degree of moisture. It is likely that these constants alter with the depth, and likewise are subject to changes from shocks.

It is a question too, whether, as is the case with loosely granular materials, friction acts (before rupture) at the same time with shear or cohesion in consolidated earth. From the interesting remarks* of Mansfield Merriman, M. Am. Soc. C. E., on internal friction, it seems probable that friction and shear exist at the same time in a solid; but, to reach sound conclusions, as he states, "further studies on internal friction and on internal molecular forces are absolutely necessary."

From the present state of our knowledge with respect to the theory and physical constants pertaining to consolidated earth, it would seem that experience must largely be the guide in dealing with it. The facts are supreme—the rational theory may come later.

Similarly, for retaining walls backed by loosely aggregated, granular materials, the facts are supreme, and, on that account, they have been presented very fully in this paper; further, a theory has been found to interpret them properly. It is true that the fresh earth, from the time that it is deposited behind a retaining wall, begins to change to a consolidated earth, from the action of rains, the compression due to gravity, and the influence of those cohesive and chemical affinities which manufacture solid earths and clays out of loosely aggregated materials, and even cause the backing sometimes to shrink away from the wall intended to support it; but it is plain that the wall should be designed for the greatest thrust that can come on it at any time, and this, in the great majority of cases, will occur when the earth has been recently deposited.

The cases which have been observed where the bank has shrunk away from the wall and afterward ruptured (after saturation, perhaps) are too few in number to warrant including in a general scheme of design, even supposing that a rational theory existed for such cases. A few remarks on the theory pertaining to the design of retaining walls may not be inappropriate. From the discussion of all the experiments referred to in this paper, the conclusion may be fairly drawn that the sliding wedge theory, involving wall friction, is a practical one for granular materials of any kind subjected to a static load. In practical design, however, vibration due to a moving load has to be allowed for; also the effect of heavy rains. Both these influences tend generally to lower the coefficients of friction and add to the weight of the filling. Mr. Baker says:

* "Mechanics of Materials," Tenth Edition, p. 381.

"Granite blocks, which will start on nothing flatter than 1.4 to 1, will continue in motion on an incline of 2.2 to 1,* and, for similar reasons, earthwork will assume a flatter slope and exert a greater lateral pressure under vibration than when at rest."

Instances of slips in railway cuttings, caused by the vibration set up by passing trains, have been given by many engineers. The effect of vibration is most pronounced near the top of a retaining wall, and is evidently greater for a low wall than for a high one. All the influences cited can only be included under the factor of safety, and the writer recommends for walls from 10 to 20 ft. in height a factor of 3. This may be increased to 3.5 for walls 6 ft. high and decreased to 2.5 for walls 50 ft. high, or those with very high surcharges. In the application, the normal component of the earth thrust on the wall, E_1, will alone be multiplied by the factor, the friction, E_1 tan. ϕ', exerted downward along the back of the wall, being unchanged. This allows very materially for a decrease in ϕ' due to rains and vibration, as well as for an increase in the thrust, due to ϕ becoming less.

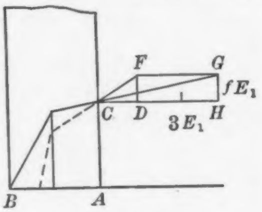

FIG. 15.

The effect is illustrated in Fig. 15, where a retaining wall is supposed to be subjected to the earth thrust, CF, making an angle ϕ' with the normal to the face, AC, of the wall. The component of CF normal to AC is $E_1 = CD$, the component acting downward along CA is represented in magnitude and direction by DF, which equals E_1 tan. ϕ'. Suppose the factor of safety to be 3, then CD is extended to H, making $CH = 3CD$; HG is drawn equal and parallel to DF; whence GC will represent the thrust, which, combined with the weight of the wall, acting through its center of gravity, must pass through the outer toe of the wall.

To see what thickness of a vertical rectangular wall corresponds to this factor of safety, 3, for $\phi = \phi' = 33° 41'$, or a natural slope of 3 base to 2 rise, let it be assumed that the weights per cubic foot of earth and cut-stone masonry in mortar are in the ratio of 2:3; then, for level-topped earth, a computation shows that, for the factor, 3, the base of the wall must be $0.34\,h$. If the earth slopes indefinitely at the angle of repose from the top of the back of the wall, and a factor 2.5 is used, then the thickness will be $0.49\,h$.

* Perhaps this may be accounted for by supposing cohesion between the blocks at rest, which is destroyed by the motion, when only friction acts.

For brick masonry in mortar, the specific weight of which is $\frac{5}{4}$ of that of the filling, the foregoing thickness would be changed to $0.37\,h$ and $0.52\,h$, respectively, h being equal to the height of the wall.

It must be noted especially, however, that if the original earth thrust, when combined as usual with the weight of wall, gives a resultant which passes outside of the middle third of the base of the wall as computed above, then the thickness must be increased, so that the resultant will at least pass through the outer middle-third limit. This ensures compression over the whole base and no opening of part of the joint under normal conditions. With regard to the thickness above of about one-third of the height, Mr. Baker states that hundreds of brick revetments have been built by the Royal Engineer officers, with a thickness of only $0.32\,h$ for a vertical wall. He advises, as the result of his own extensive experience, that the thickness be made one-third of the height for level-topped earth of average character, and that the wall be battered $1\frac{1}{2}$ in. to the foot. He states, further, that, under no ordinary conditions of surcharge on heavy backing is it necessary to make the thickness of a retaining wall on a solid foundation more than one-half the height. The thicknesses computed above agree fairly well with those recommended by Mr. Baker, and it would seem that a table of thicknesses computed on the above basis should correspond to safe walls under ordinary conditions.

It has been noted above that Equation (1), corresponding to a slope of indefinite extent, probably gives too great a thrust; besides, there are no embankments with such a slope. An embankment from 100 to 150 ft. high, supported by a low wall, may approximate the conditions assumed, but, before it is finished, the earth has consolidated to such an extent that the actual thrust is doubtless much less than the computed one. The truth is that, in nearly all back-filling of ordinary earth, the cohesive and chemical affinities commence their work very soon after the filling is deposited, and consolidation is gradually effected; so that, as has been stated, the actual thrust is often much less than is estimated in the design of the wall, where cohesive forces are neglected. In many old walls, as has been observed, the consolidation has gone so far that the backing has shrunk away from the wall altogether. It would be hazardous, though,

to allow for cohesion, in a wall backed by fresh earth, unless the surcharge was high and was a long time in building. Finally, it should be observed, that the footing of a retaining wall should be wide, and should always be tilted at such an angle that sliding is impossible.

A glance at Figs. 4, 5, and 6, will make it apparent that the Rankine and other theories differ in their results mainly because of the assumed difference of inclination of the earth thrust. In the design of walls, however, the method proposed (Fig. 15) will approximate in results those given by the Rankine theory, where, say, the earth thrust, whether inclined or not, is multiplied by the factor of safety. The writer does not advocate the middle-third limit method in design, as it gives variable factors of safety for different types of walls. Besides if the actual resultant on the base passes one-third of its width from the outer toe, there is no pressure at the inner toe, and the unit pressure at the outer toe is double the average. If vibration or other cause increases the thrust, the joint at the inner toe opens, and the pressure is concentrated too much near the outer toe. In the reinforced concrete wall, the earth thrust on a vertical plane through the inner toe is required. As this plane lies well within the earth mass, the thrust on it must be taken as acting parallel to the top slope, and its amount will be the same as that given by the Rankine theory.

Although it is highly desirable to have more precise experiments on large models in order to draw sure conclusions, yet, as far as the experiments go—those which have been analyzed and discussed in this paper—the following conclusions may be stated:

1.—When wall friction and cohesion are included, the sliding-wedge theory is a reliable one, when the filling is a loosely aggregated granular material, for any height of wall.

2.—For experimental walls, from 6 to 10 ft. high, and greater, backed by sand or any granular material possessing little cohesion, the influence of cohesion can be neglected in the analysis. Hence, further experiments should be made only on walls at least 6 ft., and preferably 10 ft., high.

3.—The many experiments that have been made on retaining boards less than 1 ft. high, have been analyzed by their authors on the supposition that cohesion could be neglected. This hypothesis is so far from the truth that the deductions are very misleading.

4.—As it is difficult to ascertain accurately the coefficient of cohesion, and as it varies with the amount of moisture in the material, small models should be discarded altogether in future experiments, and attention should be confined to large ones. Such walls should be made as light, and with as wide a base, as possible. A triangular frame of wood on an unyielding foundation seems to meet the conditions for precise measurements.

5.—The sliding-wedge theory, omitting cohesion but including wall friction, is a good practical one for the design of retaining walls backed by fresh earth, when a proper factor of safety is used.

As the subject of pressures on the roof and sides of a tunnel lining has received much attention of late, the writer has concluded to extend this paper, so as to give a development of a theory, based on the grain-bin theory of Janssen, but modified to include the cohesive or shearing resistances of the earth in addition to the frictional resistances.

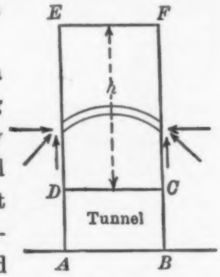

FIG. 16.

Fig. 16 is a vertical transverse section of a tunnel, *ABCD*, and the earth, *DCFE*, extending over it h feet. If this tunnel has been driven by the use of a shield or poling boards, the ground will tend to settle over it, and part of the weight of *DCFE* will be sustained by cohesion and friction (resulting from the lateral thrust) exerted along the sides, vertically upward. The earth will probably arch itself, or form a series of domes superposed one upon the other, but the external forces acting on such domes will be the same as those acting on a corresponding horizontal lamina, and the theory, given in full in the Appendix, begins with the considerations pertaining to the equilibrium of such a lamina.

If there was no settlement of the earth, *DCFE*, in relation to *DC*, then the vertical pressure per square foot on *DC* would be $w\,h$ (w being the weight of a cubic foot of the earth in pounds), but, as most of the weight of *DCFE* is carried by the sides, in case of sufficient settlement, the vertical unit pressure, V, on *DC*, will be much less than $w\,h$. Also, the lateral unit pressure, L, at the level, *DC*, will be much less where settlement occurs. From the equations for V and L, given in the Appendix, the diagrams, Figs. 17 and 18, have been constructed.

In both diagrams, the weight of the earth was taken at $w = 90$ lb. per cu. ft., and the cohesion of the earth at $c = 100$ lb. per sq. ft. In Fig. 17, $\phi = 45°$ and the curves for V and L were laid off for a width of tunnel, b, of 15 ft. and also for 30 ft. In Fig. 18, $\phi = 30°$, and curves are given for V and L, also for $b = 15$ ft. and 30 ft. for various heights, h.

It will be perceived, in both figures, that when certain heights are attained, both V and L cease to increase perceptibly, so that such values may be taken as corresponding to h indefinitely large.

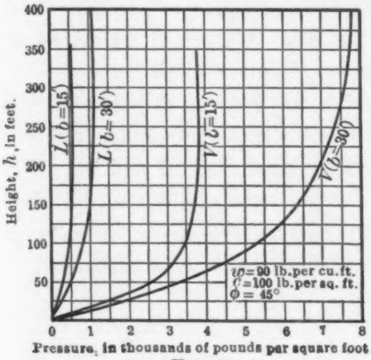

FIG. 17.

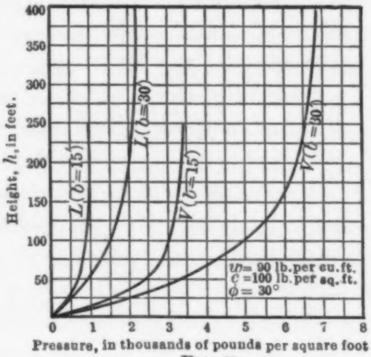

FIG. 18.

A simple way of deriving these extreme values is given in the Appendix. The values of ϕ, b and w have been taken here the same as those used by Mr. Meem, in framing his table of pressures,* which may be supposed to embody, in part, practical experience. The results found from Figs. 17 and 18 by the writer, for a depth of covering of several hundred feet, are uniformly much larger than those given by Mr. Meem. Are they too large for safety? In answering this question, it must be remembered that, of the weight of earth directly over the tunnel, all has been transferred to the sides that it was possible to transfer, for the coefficients of friction and cohesion given. We know scarcely anything of the cohesion coefficients, so that the value assumed, $c = 100$ lb. per sq. ft., may not be near the truth. Certainly it must appear plain from this discussion that the values of c and ϕ must be better known, for all kinds of earth, before reliable results can be attained. The results are submitted for discussion, in the hope that engineers will give their experience relative to the pressures

* *Transactions*, Am. Soc. C. E., Vol. LXX, p. 387.

realized in the timbering of tunnels, particularly through sand or earth not thoroughly consolidated.

The value of V, in Figs. 17 and 18, is the average vertical unit pressure at the top of the tunnel. Experiments on grain bins lead to the inference that the pressure at the middle of the roof is greater than that at the sides, but no law of variation can be stated.

The lateral unit pressure on the vertical sides of the tunnel lining at the top is given by the equation for L, or by the corresponding diagram. The variation in this lateral pressure over the sides of the tunnel cannot be easily formulated, as so much of the weight of the earth, directly over the tunnel, has been transferred by a kind of arch action to the sides. Experience would better speak here.

Table 5 gives the values of V and L for $h = 40$ ft. The figures in Columns M are taken from Mr. Meem's table, previously referred to; those for Columns C are from the diagrams, Figs. 17 and 18.

In quoting Mr. Meem's figures, the writer must not be understood as endorsing in any way his theory; but the results are of interest as embodying the conclusions of a practical engineer of large experience.

TABLE 5.

b, in feet.	ϕ.	V, IN POUNDS PER SQUARE FOOT.		L, IN POUNDS PER SQUARE FOOT.	
		M.	C.	M.	C.
15	45°	1 485	2 300	405	300
15	30°	1 035	2 100	540	600
30	45°	3 240	2 800	450	400
30	30°	2 325	2 600	450	750

If the height, h, of earth covering is 200 or 300 ft., the values given by Figs. 17 and 18 are much larger than those given in Columns M, which presumably represent Mr. Meem's pressures for any height greater than 40 ft.

In saturated earth, it has been customary, perhaps, to regard the earth as if it were gravel composed of solid spheres, like marbles, so that the water has free access in any direction. Thus, in the case of a retaining wall backed by such material, the water has full access practically to every part of the wall, and the wall is subjected to the full water pressure corresponding to its depth. It is likewise subjected to a thrust from the earth, corresponding to c and ϕ, for the saturated material, but with a weight per cubic foot equal to that of the earth in air less the buoyant effect of the water. Thus,

if a cubic foot of the porous earth, in air weighed 90 lb., and if the voids were 40%, then 1 cu. ft. of earth contains 0.6 cu. ft. of solids and the buoyant effect of the water is the weight of an equal volume of water or $0.6 \times 62.5 = 37.5$ lb. Hence, the weight per cubic foot of earth in water is $90 - 37.5 = 52.5$ lb.

Similarly, for the pressures, V and L, at the top of a tunnel, $w = 90$ must be replaced by 52.5, and c and ϕ must be found for the saturated material and these values substituted in Equations (5) and (6) of the Appendix. To these pressures must be added the corresponding water pressures for the full height of water, supposing it to have free communication everywhere, as in the case of the gravel filling. However, with sand, or earth with much fine material, the pores are more or less clogged up and there is perhaps intimate contact of a part of the earth with the roof of the tunnel, so that the water cannot get under it to produce a lifting effect, and if such intimate contact is found along any horizontal or vertical section, of the earth on either side of the section, it is plain that the buoyant effort of the water on a cubic foot of material will be much diminished.

Mr. Meem deserves great credit, not only for calling attention to this, but especially for performing certain experiments to prove it.* The experiments were on sand, and only on a small scale, but the practical conclusion drawn from them is that the water pressure transmitted through sand having 40% voids is diminished about 40% in intensity. This occurs for a depth of only a few inches of sand, and presumably the diminution would be greater for sand several feet in depth. Of course, before definite values can be stated, experiments on a large scale should be made on every kind of material usually met; but, as a numerical illustration of the application, for the diminution mentioned—which is assumed to extend through the mass—it is seen that, in the examples of the retaining wall and also the tunnel, the weight per cubic foot of the earth in water must now be taken at $90 - (0.4 \times 0.6 \times 62.5) = 75$ lb. per cu. ft.

This value replaces the w in Equations (5) and (6) from which the V and L for the top of the tunnel are found. To these values, add $0.4 \times 62.5\ h_0$, for the water pressure, where the surface of the water extends a height, h_0, above the top of the tunnel. Similarly, in the case of the retaining wall, add 0.4 of the full water thrust on the wall to that given by the earth, weighing only 75 lb. per cu. ft.

* *Transactions*, Am. Soc. C. E., Vol. LXX, pp. 365-368.

As a numerical illustration, take $b = 15$ ft., $\phi = 45°$, w in air $= 90$ lb., $h = 40$ ft., $h_0 = 60$ ft.; but we must now replace w by the weight in water, 75 lb., as found above. The values of V and L are now found, by Equations (5) and (6) (Appendix), to be 1 917 and 246 lb. per sq. ft., respectively, for the saturated earth alone. To these values add $0.4 \times 62.5 \times 60 = 1\,500$ lb. per sq. ft., water pressure, giving a total of 3 417 and 1 746 lb. per sq. ft., respectively, for the vertical and horizontal unit pressures at the top of the tunnel lining.

In connection with this subject of underground pressures, it may not be inappropriate to make some concluding remarks on the maximum vertical pressures to which culverts may be subjected.

Let Fig. 16 now represent a longitudinal vertical section along the axis of a road embankment, built over an arch culvert or box-drain, *ABCD,* the line, *CD,* passing through the summit of the arch or the top of the covering stone of the box-drain, and the lines, *AD* and *BC,* coinciding in part with the exterior sides of the abutments.

There is a horizontal thrust of the earth on the medial plane, *CDEF,* acting at right angles to the plane of the paper, which tends to distribute the weight of the central portion partly toward the sides; but, ignoring this, it is seen that, if the earth everywhere settles uniformly, the maximum pressure per square unit at the top of the culvert is $w\,h$, and the total vertical pressure on the culvert is the weight of the earth vertically above it.

If, however, the earth outside the abutment walls settles more than the walls (a case which may occur), then part of its weight, and that of the earth vertically above it, will be transferred, through friction and cohesion, along the planes, *AE* and *BF,* to the culvert, and thus the vertical pressure on the top of the culvert will be greater than in the first supposed case; but, if the reverse obtains, or if the culvert settles more than the earth outside the lines, *AD* and *BC,* or if the arch or covering stone descends in the middle relatively to the abutments, then part of the weight of the earth vertically over the culvert is transferred to the sides. For a comparatively rigid arch, the settlement is perhaps not enough to warrant us in making the maximum unit pressure less than $w\,h$. Exactly what settlement would warrant the use of the theory set forth in the Appendix it is impossible to say. If the unit pressure is taken as $w\,h$, we can rest assured that in most cases the real pressure is materially less.

APPENDIX

In the experiments of Jamieson and Pleissner on the pressures in deep grain bins*, the ratio, k, of the lateral unit pressure, L, on a vertical plane to the vertical unit pressure, V, on a horizontal plane, was found by Pleissner to vary from 0.3 to 0.5 and by Jamieson to equal 0.6, for wheat in wooden bins of various sizes.

This ratio, $k = \frac{L}{V}$, increases somewhat with the depth of the grain, but the increase is slight after a depth of from $2\frac{1}{2}$ to 3 times the width or diameter of the bin is reached.

It is recognized that the proper value of k, for a particular case, can only be determined properly by experiment, but it is interesting to note that, by the theory of earth pressure of an unlimited granular mass, level at the top, the ratio of the lateral to the vertical unit pressure, at any point in the mass, is, $k' = \tan.^2\left(45° - \frac{\phi}{2}\right)$, and that this varies from 0.361 to 0.406, as ϕ, the angle of repose, varies from 28° to 35°, the values of ϕ for wheat, given in some of the experiments. Further, by reference to Jamieson's experiments on a model bin of smooth steel, 1 ft. in diameter,† filled with sand, for which $\phi = 34°$, $k' = \tan.^2\left(45° - \frac{\phi}{2}\right) = 0.283$, we find the experimental value of k to equal k' exactly for a height of sand of 2.5 ft., the value at 6 ft. and upward being 0.33.

The theory of bin pressure is utterly different from the ordinary theory of earth pressure in an unlimited granular mass; but it is seen that the latter may be of some use in furnishing a value of k when experimental values are lacking, as in the case of various kinds of earth, both granular and more or less consolidated.

An equation for L, for an unlimited mass of earth, level at the top and having a coefficient of cohesion, c, has been given by Scheffler,‡ and is as follows:

$$L = w\,h \tan.^2\left(45° - \frac{\phi}{2}\right) - \frac{c \cos.\,\phi}{\cos.^2\left(45° - \frac{\phi}{2}\right)} \quad \ldots\ldots\ldots(1)$$

* Given in detail in "The Design of Walls, Bins and Grain Elevators," by Milo S. Ketchum, M. Am. Soc. C. E.

† Ketchum's "Walls, Bins, and Grain Elevators," Fig. 171.

‡ "*Traité de Stabilité des Constructions*," p. 292; see also Remark at end of Appendix.

Here w = weight per cubic unit of earth,

h = depth below the level-topped, unlimited mass,

L = unit lateral pressure on a vertical plane at the depth, h.

This reduces to the usual formula when $c = 0$.

It is seen from Fig. 19, if we lay off at the depth, h, $BC = w\,h \tan.^2 \left(45° - \frac{\phi}{2}\right)$, that the horizontal ordinates of the triangle, ABC, measure the values of the first term of the right member of Equation (1). The second term, $\dfrac{c \cos.\phi}{\cos.^2\left(45° - \frac{\phi}{2}\right)}$, is constant, and is represented by the horizontal ordinates of the rectangle, $ABGF$. Thus the value of L at the depth, h, is represented by $BC - BG = CG$.

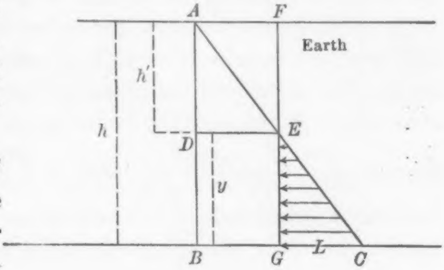

FIG. 19.

At D, $L = DE - DE = 0$; but, above D, the equation is inapplicable, for negative values of L, corresponding to tension along AD, are inadmissible; hence, above D, we must write $L = 0$.*

In Equation (1), for $h = h' = AD$, $L = 0$,

$$\text{therefore,}\quad 0 = w\,h' \tan.^2 \left(45° - \frac{\phi}{2}\right) - \frac{c \cos.\phi}{\cos.^2\left(45° - \frac{\phi}{2}\right)} \quad \ldots\ldots\ldots (2)$$

Solving this for h',

$$h' = \frac{c}{w}\,\frac{2 \cos.\phi}{1 - \sin.\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For given values of c, w, and ϕ, having computed h', we have, on subtracting Equation (2) from Equation (1),

$$L = w\,(h - h') \tan.^2 \left(45° - \frac{\phi}{2}\right) \quad \ldots\ldots\ldots\ldots\ldots (4)$$

which is true only for $h = h'$ or $h > h'$. When $h < h'$, $L = 0$.

If we put $y = (h - h')$, the equation reduces to the ordinary form, and thus the center of pressure of the thrust on AB (or DB) acts at $\frac{1}{3}\,y$ above B, and its amount is $\frac{1}{2}\,w\,y^2 \tan.^2 \left(45° - \frac{\phi}{2}\right)$.

* Scheffler has not noted this fact, and consequently some of his deductions are open to objections. His theory, involving cohesion, is the only one the writer has seen.

Call V, the vertical unit pressure at the depth h,

V', " " " " " " " h',

therefore $V = w\,h = w\,(y + h')$,

$V' = w\,h'$;

and, putting $k = \tan.^2\left(45° - \frac{\phi}{2}\right)$, we have, from Equation (4),

$$L = w\,(h - h')\tan.^2\left(45° - \frac{\phi}{2}\right) = (V - V')\,k.$$

Next consider the case of a tunnel of width, b, and length, l, which has been driven by shield or by use of timbering, so that an appreciable settlement of the roof occurs; then the weight of the earth vertically over the tunnel is partly carried by the adjacent walls of earth, by friction and cohesion, and it would seem that such walls can be supposed to take the place of vertical grain bin walls, and that the theory of bin pressures corresponding may be made to apply. The theory that will be developed, which includes the influence of cohesion, is simply a modification of that used in developing Janssen's formula, as given by Mr. Ketchum,* and, for a ready comparison of results, his notation will be used.

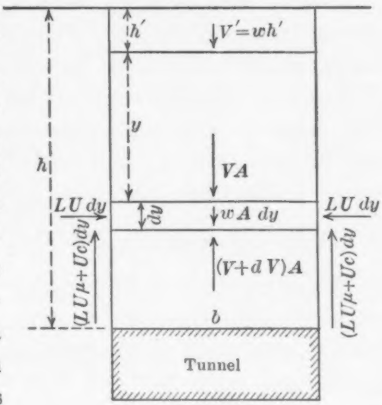

FIG. 20.

In Fig. 20, let h be the distance from the roof of the tunnel to the surface of the earth. Compute h' from Equation (3) and lay it off from the top down.

Let V = vertical unit pressure at depth, $y + h'$;

L = lateral " " " " "

w = weight of earth per cubic unit;

ϕ = angle of repose of earth;

$\mu = \tan.\,\phi$ = coefficient of friction of earth on earth;

$A = b\,l$ = area of horizontal section of tunnel;

$U = 2\,(b + l)$ = perimeter of section; and

c = cohesive force of earth per square unit.

* "The Design of Walls, Bins and Grain Elevators," Chapter XVI.

The foregoing equation, $L = (V - V')\, k$, will be used as a semi-empirical formula to express the relation between L and V, but now V is no longer equal to $w\,(y + h')$ but at present is unknown. As before, $V' = w\, h'$.

Properly, k should be determined by experiments, but, from lack of such experiments, it will be computed from the formula above,

$$k = \tan.^2 \left(45° - \frac{\phi}{2}\right).$$

Consider now the conditions of equilibrium of a horizontal slice of earth of depth, dy, the weight of which is $A\, w\, d\, y$.

The top surface, at the depth, $y + h'$, is acted on by the force, VA, acting downward, and the bottom surface by the force, $(V + d\, V)\, A$, acting upward. The total lateral force acting on the area, $U\, d\, y$, is $L\, U\, dy$ and this causes a frictional force of $L\, U\, \mu\, dy$, acting upward. The cohesive or shearing resistance on the area, $U\, dy$, acts upward, and its amount is $U\, dy\, c$. Placing the sum of the vertical forces acting on the slice equal to zero.

$$V\, A + A\, w\, dy - (V + d\, V)\, A - (L\, U\, \mu + U\, c)\, dy = 0.$$

In reality, an arch or dome of the earth should be considered in place of the horizontal stratum, but the result is the same, because the same vertical forces act in either case. Simplifying the above equation, and dividing by A,

$$d\, V = \left[w - (L\, \mu + c)\, \frac{U}{A}\right] dy.$$

Putting $R = \dfrac{A}{U}$, and $L = (V - V')\, k$,

$$\frac{d\, V}{dy} = w - \Big[(V - V')\, k\, \mu + c\Big] \frac{1}{R}$$

$$= w - \frac{k\, \mu\, V}{R} - \frac{c - V'\, k\, \mu}{R} = w - n\, V - q.$$

on placing, $n = \dfrac{k\, \mu}{R}$, $q = \dfrac{c - V'\, k\, \mu}{R}$.

It follows that, $$\frac{-\, n\, d\, V}{w - n\, V - q} = -\, n\, dy$$

therefore $\log.\, (w - n\, V - q) = -\, n\, y + C.$

When $y = 0$, $V = V'$,

$$\text{therefore log. } (w - n\, V' - q) = C,$$

$$\text{log. } \frac{w - n\, V - q}{w - n\, V' - q} = -\, n\, y,$$

$$\text{and } \frac{w - n\, V - q}{w - n\, V' - q} = e^{-n\, y},$$

where $e = 2.71828 \ldots$, the Napierian base.

Solving for V,

$$V = \frac{w - q}{n}\left[1 - e^{-n\, y}\right] + V'\, e^{-n\, y}.$$

Substituting the values of n and q, we have,

$$V = \frac{1}{k\,\mu}\left[R\, w - (c - V'\, k\,\mu)\right]\left(1 - e^{-\frac{k\,\mu}{R}\, y}\right) + V'\, e^{-\frac{k\,\mu}{R}\, y} \ldots (5)$$

in which it must be remembered that,

$$V' = w\, h';\; h' = \frac{2\, c \cos.\, \phi}{w\,(1 - \sin.\, \phi)}.$$

The lateral thrust is now given by,

$$L = (V - V')\, k \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

To get the pressures, V and L, at the top of the tunnel, replace y by $(h - h')$.

The weight of the upper stratum, of depth h', is in part sustained by the cohesion of the sides, but as h' is generally small, this cohesive force can be neglected, as was done above.

Equations (5) and (6) reduce to the ordinary bin formulas of Janssen, when $\mu = \mu'$, $c = 0$, and therefore $V' = 0$, $h' = 0$. The modification due to these terms is generally small, unless c is very large.

For large values of y, $e^{\frac{-k\,\mu}{R}\, y}$ is small, and as y increases indefinitely, V approaches as a limit the value

$$\frac{1}{k\,\mu}\left[R\, w - (c - V'\, k\,\mu)\right].$$

This expression may be derived independently, and is of practical value when a very high surcharge is considered.

Referring to Fig. 20, it is evident that the maximum limit of V would be realized if the weight of any horizontal lamina is entirely

held up by the friction and cohesion of the sides; for then, for all lower slices, V and L remain the same.

$$\text{Therefore } A\ w\ dy = (L\ U\ \mu + U\ c)\ dy$$

$$A\ w = (V - V')\ k\ U\ \mu + U\ c,$$

$$\text{therefore } V = \frac{1}{k\ \mu}\Big[R\ w - (c - V'\ k\ \mu)\Big],$$

as given above.

As seen, such a state is not exactly realized, but is practically true for great depths.

For a long tunnel, the perimeter of the section, U, can be taken as $2\ l$, whence,

$$R = \frac{b\ l}{2\ l} = \frac{b}{2}.$$

This value was used in all the computations.

As a numerical illustration of the use of Equations (5) and (6), suppose a tunnel, $b = 15$ ft. wide, and, for the earth covering let $\phi = 45°$; therefore $\mu = \tan.\ \phi = 1$, $w = 90$ lb. per cu. ft., and $c = 100$ lb. per sq. ft.

We deduce $k = \tan.^2\ (45° - 22\tfrac{1}{2}°) = 0.172$, $R = \dfrac{b}{2} = 7.5$, $h' = \dfrac{2\ c \cos.\ \phi}{w\ (1 - \sin.\ \phi)} = 5.4$ ft., therefore $V' = w\ h' = 486$ lb., $(c - V'\ k\ \mu) = 100 - 83 = 17$.

Equations (5) and (6) readily reduce now to

$$V = 3\ 830\ (1 - e^{-0.023\ y}) + 486\ e^{-0.023\ y},$$

$$L = (V - 486) \times 0.172.$$

These formulas give the vertical and horizontal unit pressures at the top of the tunnel when,

$$y = h - h'.$$

In computing the values of V and L for various depths of earth covering, h, a short table of hyperbolic logarithms is a convenience. The curves given by the equations above are shown on Fig. 17.

An additional note with respect to Equation (1) may not be inappropriate. Scheffler, in deriving this equation, considered the conditions of equilibrium of an infinitesimal wedge of earth at the depth, h. It was found that the horizontal pressure at the depth, h', given by Equation (3), was zero, and it was assumed by the writer that there was no pressure on a vertical plane for a less depth. Thus, in

Fig. 21, there is no horizontal pressure on the plane, DE, where $DE = IC = h'$; consequently, the weight of the wedge, EDB, is supported entirely by the normal reaction of the plane, DB, with the cohesion and friction acting along it.* To deduce Q, the total earth pressure on the vertical plane, AC, it is then admissible to treat the prism, $ADEC$, as the prism of rupture, the surface of rupture consisting of the plane, AD, making the angle α with the vertical and the plane, DE. Therefore W, the weight of the prism, $ADEC$, is in equilibrium with Q, N, and $N\,f + K$, where N is the normal reaction of AD, $f = \tan.\ \phi$, and $K =$ the total cohesion on $AD = c\ .\ AD$.

On balancing components parallel and perpendicular to the plane, and then following familiar methods, it can be shown that the true value of Q corresponds to $\alpha = \dfrac{90 - \phi}{2}$, and that this value is,

$$Q = \frac{w}{2}(h^2 - h'^2)\tan.^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right) - \frac{c\,(h - h')\cos.\ \phi}{\cos.^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)}.$$

The derivative of this, with respect to h, gives the intensity, L, at the depth, h, exactly the same as Equation (1), and the subsequent deductions hold. Thus the fundamental Equation (1), according to the interpretation given, is seen to correspond to a prism of rupture, $ADEC$, which is a little nearer the true one, having a curved surface of rupture, than the wedge, ABC.

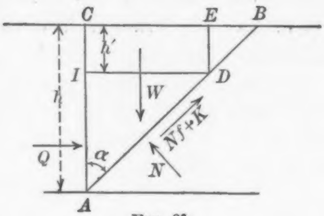

FIG. 21.

The above refers to the pressure on a vertical plane of a mass of level-topped earth of indefinite extent; but suppose that AC is the back of a retaining wall, and that a slight movement downward of the prism of rupture is imminent; then, if the earth along the plane,

* It may be well to remark here, that for cohesive earth, it has been proved, both theoretically and experimentally, that the surface of rupture is curved, and not a plane, as the theory assumes. However, assuming it to be a plane, and considering successive wedges of rupture of different heights, the bases of which lie on the same plane, it can be easily shown that certain of the upper wedges can be sustained by cohesion alone, and that the coefficient of cohesion required for stability varies from 0 at the surface to its maximum value at a certain depth, h_1. Below this depth, friction in addition to cohesion is exerted, and stability is assured if we suppose the friction coefficient to increase from 0 at $h = h_1$ to its maximum value, tan. ϕ, at some depth, $h = h'$. Below this depth, on the plane of rupture, the maximum values of both coefficients are exerted. Now, the ordinary wedge theory assumes, for simplicity, that these coefficients are constant all along the plane of rupture, which may be true at the instant of rupture, but not for a stable mass. It is possible, too, that rupture may be progressive, starting at the bottom.

DE, can exert sufficient tension, the mass, *ADEC*, in descending, may drag down the wedge, *DEB*, with it, so that the full friction and cohesion along *DB* will be added to that along *AD*. In other words, the prism of rupture must now be taken as the wedge, *ABC*; hence, the value of Q corresponding is given by the equation above, on making $h' = 0$, as this introduces, in the first equations for equilibrium, the fact that the prism of rupture is now the wedge, *ABC*.

It is only one step farther to find the greatest height at which the vertical face of an open trench will stand for given coefficients, c and f. On making $Q = 0$ in the equation for Q above when $h' = 0$, we find, after reduction,

$$h = \frac{4\ c \cos. \phi}{w\ (1 - \sin. \phi)},$$

a value which has been quoted elsewhere in this paper. It is double the value for h' given by Equation (3). The reason for this, though, is now evident; for the last equation follows as a consequence of assuming that the full cohesive and frictional resistances along *DB* were exerted; whereas Equation (1) ignores them.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

NOTES ON THE BAR HARBORS AT THE ENTRANCES TO COOS BAY, AND UMPQUA AND SIUSLAW RIVERS, OREGON.

Discussion.*

By Lewis M. Haupt, M. Am. Soc. C. E.

Lewis M. Haupt, M. Am. Soc. C. E. (by letter).—In this interesting paper Mr. Tower has added some material facts which emphasize the importance of a thorough consideration of all the forces which cooperate to effect changes at alluvial inlets, and directs attention to the influence of the flood-tidal currents as being an important factor, if not the main one, in causing troublesome shoals. Mr. Haupt.

Lest too much stress be unduly laid on a single factor, it is well to note that the literature on this subject is prolix, and that many physical hydrographers and maritime engineers have directed specific attention to the influence of tidal currents and the paths which they take in approaching the tidal estuaries or any breach in the ocean littoral. Rear-Admiral Davis, in "The Law of Deposit of the Flood Tide: Its Dynamic Action and Office,"† Sir John Coode, on the "Chesil Banks," and Wheeler, Mitchell, Hilgard,‡ and many others have definitely directed attention to this factor, and the discussions are of great interest to the profession, because so much is involved in the

* This discussion (of the paper by Morton L. Tower, M. Am. Soc. C. E., printed in *Proceedings* for November, 1910, and presented at the meeting of December 21st, 1910), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Smithsonian Contributions, Vol. III, 1851, where "It was laid down as a fundamental principle, that the deposits on the ocean border are only made by the current of the flood tide." See also "Discussion on the Dynamic Action of the Ocean in Building Bars," by the writer, Am. Phil. Soc., March, 1889.

‡ Smithsonian Report, 1874, p. 219. Also letter to the writer, May 20th, 1880, stating "You are entirely correct, as it is the unceasing activity of the flood that produces the forms so characteristic of harbor entrances." See *ante*, Am. Phil. Soc., 1889.

Mr. Haupt. cost of harbor bar improvements and their maintenance in a fixed position.

When the writer was stationed on the Texas Coast, in 1869, as the United States Engineer of the Fifth Military District, his attention was directed to the delays in crossing the bars and the necessity of lightering passengers and live stock off shore. This led him to devote his attention to these specific problems, with the result that he designed a system whereby a part of one jetty, properly placed, might control the resultant forces in such manner as to remove the bar automatically and create a self-maintaining channel, and Congress was so well satisfied with the merits and economies of this device, that in 1902 a test of its efficiency was ordered to be made at Aransas Pass, Texas, where all previous efforts had failed. After the removal of the former obstacles, and before the original design was completed in its entirety, the feeble 14-in. tide had scoured out the channel to depths of from 20 ft. to more than 26 ft., the original depth being from 5 to 8 ft., without any dredging whatever. This work was designed to control the sand driven on the bar by the "flood-component" of the external forces, and not primarily to concentrate the ebb, as was the prevailing custom, by two jetties, which merely prolong the slope, increase the friction, reduce the tidal ingress, and throttle the inlets so as to require frequent dredging to create and maintain the channel.

In many cases the jetty may be detached from the shore and be placed so as merely to connect the deep water on the outer and inner slopes of the bar. It should lie between the proposed channel and the source of the resultant drift and be concave to the channel, thus creating a reaction which will effectually prevent the deposit of material on the protected portion of the bar.

The author calls attention to the danger of the structure being undermined if placed too near the channel, but at Aransas Pass this has been prevented by a suitable apron laid along the toe of the work, which has reduced its deterioration to only about 2 per cent.

This well-known and much-discussed case is cited as furnishing the best evidence of the importance of the questions raised by Mr. Tower, and as emphasizing the great economy which may be secured from an intelligent application of the available resources of Nature, particularly as the same site, subject to the same forces, furnishes an admirable illustration of the comparative results of the two systems, both before and after the reaction jetty was partly built. The previous work, after nearly 20 years, and at a cost of more than $500 000, was abandoned with "insignificant results."

The reaction jetty, only partly finished, produced immediate scour, and soon after the removal of the old, obstructing works, cut the 20-ft. channel through, without advancing the bar, at a less cost to the Government than the former work; and when it was decided to

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attempt to deepen the channel, in 1907, before it had reached a condition of equilibrium, by closing the tidal opening and building a second jetty, the channel shoaled to less than 14 ft. immediately. In consequence it has become necessary to appropriate more than $3 000 000 in the effort to recover the depths by jetty extensions, dredging, and other works, and for the protection of the adjacent islands from erosion.

Had the general principles of the utilization of the littoral forces and the control of the bar-building drift been duly recognized when the South Pass was being opened, the late James B. Eads, M. Am. Soc. C. E., is reported to have said that he would not have built his west jetty, which has been largely buried under the deposits created by that on the east bank.

Incidentally, the closure of the Cumberland Sound, which required an emergency appropriation of $500 000 to remove a portion of the south jetty, and the effort to open a channel across the bar south of both jetties, might have been avoided, while the subsequent building up of the windward or north jetty at its outer end enabled the natural forces, aided by dredging, to restore a fairly good, normal channel.

The use of the two jetties at the Southwest Pass, aided by constant dredging and the extension of the bar far beyond the ends of the work, might have been saved by the judicious application of the same forces which had created an excellent channel in the Pass above with depths of from 40 to 100 ft., in a channel 1 300 ft. in width, without cost, and had also added some 150 sq. miles of rich, fast land to the territory of the State and Nation through crevasses now being closed.

The attempt to open a deep-water harbor at Cold Spring Inlet, N. J., by two jetties about 700 ft. apart and 1 mile long, which has resulted in a channel 6 ft. deep at mean low water, at a cost of more than $1 000 000, might have been saved. Many other instances of the practical application of the general principles enunciated in the paper might be cited, but these few should suffice to point the moral as to the great possibilities of securing better results at less cost and without artificial aid, by a recognition of the fact that sand is heavier than water, that the breakers are the most potent agencies in transporting it, and the flood-tide the great propelling force, while, when once deposited, a current unaccompanied by breakers is impotent to disturb it unless a reaction or eddy is set up by some resisting medium adjusted so as to cause erosion.

Furthermore, it would seem to be pertinent at this time to direct the attention of Members of this Society to the paramount importance of giving due weight to these dynamic forces, which are so vital in the restoration of our former commercial supremacy by re-opening the channels to the sea, to state that the plan now proposed for the creation and maintenance of the entrance to one of the greatest engineering

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enterprises on earth does not seem to have been adapted to protect the channel from the action of the off-shore forces. These must create an extensive shoaling directly across the path of navigation, and the extensive breakwaters will not prevent but rather augment it, requiring constant and expensive dredging to remove. This may be avoided in the first instance at much less cost.

Much might be added as to the cyclic movements of the ebb thalweg over the outer bars of entrances and the necessity of arresting the forces which produce them, that the location of the channel may be made permanent; but the whole problem has been presented so fully in previous discussions in the *Transactions* of the Society and elsewhere, that it will suffice to refer to them for further information.

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PAPERS AND DISCUSSIONS

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THE VALUATION OF PUBLIC SERVICE CORPORATION PROPERTY.

Discussion.*

By Messrs. Fred Lavis, Charles H. Higgins, S. D. Newton, William V. Polleys, C. P. Howard, J. E. Willoughby, Henry C. Adams, Carl C. Witt, R. A. Thompson, Charles H. Ledlie, and William G. Raymond.

Fred Lavis, M. Am. Soc. C. E.—The author states that his paper is confined to "a discussion of the methods which should be used in arriving at a correct figure of cost of reproduction and depreciation," and that "it does not take up questions involving the propriety of those figures when reached." In so far as this is concerned, it is probably the most complete compilation of the available information on this phase of the subject which has yet appeared in print. The author "refuses to recognize that the consideration of the so-called intangible values has any place in a physical valuation." As, however, there exists such a widespread feeling, especially among those interested in railroads, that physical valuations, for any purpose whatever, are absolutely useless, because these intangible values are not or cannot be included, it does not seem out of place to refer to this phase of the subject at this time, and more especially in view of the fact that many persons, the prominence of whose position entitles them to consideration, have taken this point of view very recently, and their remarks have received considerable publicity. Not more than two weeks ago, Judge Lovett, the head of the Harriman System, expressed the opinion that the theory of valuing railroad property by trying to Mr. Lavis.

*This discussion (of the paper by Henry Earle Riggs, M. Am. Soc. C. E., printed in *Proceedings* for November, 1910, and presented at the meeting of January 4th, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lavis. determine the cost of reproduction was utterly impractical. It seems important, therefore, that we, as engineers, interested in having the question properly understood, should be careful, in referring to valuation, to make it plain that other features besides the value of the physical property are to receive due consideration. The speaker, therefore, proposes to examine some of the arguments advanced by the opponents of valuation to see if the objections most generally brought forward are insuperable.

Some critics of valuation go so far as to say that engineers cannot make a close valuation of even the purely physical property. For instance, Mr. W. H. Williams, Vice-President of the Delaware and Hudson Company, in a paper on this subject,* states that:

"No engineer in estimating on the several important items of construction work for the year will come within 10 per cent. of the total aggregate cost. Many of the more important items are frequently underestimated 25 to 50 per cent."

He cites, as an especially good illustration, the Panama Canal, the original estimate of the cost of which was $140 000 000, though the present estimate is $300 000 000. Almost everyone who has kept in touch with that subject knows why the Panama Canal has cost more than the original estimates, and that the greater cost is no reflection on the judgment of the engineers who made such estimates. One cannot always foresee what changes in plans may be made before construction is completed, and would hardly expect the estimates of the cost of a railroad to be adequate if they were made for a single-track road and a double-track was built. In any event, there is a vast difference in estimating the cost of an engineering work already completed and one which has yet to be started, the difference being largely in favor of a closer estimate of the completed work.

Limitations are often placed on engineers, in connection with work they do, which are afterward forgotten. The speaker was asked not long ago to prepare a report in connection with the valuation of a large railroad property. The time within which the results were required was very limited, and the methods used in the valuation necessarily had to be a combination of the inventory method and reliance, in a great many matters, on the judgment of those making the appraisal. Undoubtedly the result obtained was entirely adequate for the purpose for which it was required, but would hardly stand if an attempt were made to use it as a basis for an argument before a Court of law or a public service commission, though it would not be beyond the range of the experience of many engineers, to have a matter of this kind brought forward some time in the future as an absolute statement of fact, with no reference to the way in which the work was done.

* *Electric Railway Journal*, January 8th, 1910, p. 76.

It is inevitable, of course, that engineers will differ in their opinions as to some details of methods of making an inventory of the property of a railroad or other public service corporation, and also as to exactly what unit prices should be applied, but in general it is safe to say that any engineer of proper experience and training can make a satisfactory appraisal of the value of the physical property of a railroad, and that if two or more such competent fair-minded engineers, unhampered by any consideration of the purpose for which it is to be made, should make such an appraisal, the variation in the result would be so small as to be negligible. The speaker, however, does not entirely agree with the author, that the purpose for which the appraisal is to be used, should be entirely ignored by those who are making it. There can be little doubt as to the propriety of using a properly made physical valuation as a basis for taxation, or as information for the owners, although there may be some as to the methods whereby the so-called intangible values are to be determined in these cases, or even whether they should be considered at all. The greatest difference of opinion arises when an attempt is made to regulate the issue of stocks and bonds, or to fix the rates which should be charged for transportation, on the basis of a physical valuation. Mr. Lavis.

Arguments for and against rate regulation revolve in a circle, and, apparently, there is no starting point which will satisfy every one. The Courts have ruled that the railroads are entitled to such rates as will enable them to earn a fair return on the value of their property; the railroads claim that the only way to determine this value is on the basis of the earning capacity; that is, one side claims that the rates must be based on the value and the other that the value should be based on the rates. It is evident, however, by this time that the railroads must submit to regulation, therefore a way must be found to break into the circle, and it would seem to be incumbent on them to direct their energies along lines which will tend to make such regulation fair and just rather than to oppose it entirely. There is little claim that unduly large dividends are paid, but there is a feeling in the mind of the public that the railroads are over-capitalized. Is it not possible, therefore, to break into the circle at this point, and decide, by means of a proper valuation, as to the fairness or otherwise of the capitalization? The objection to this, on the part of the railroads, is that the value of the purely physical elements is by no means the whole value of their property, but that something should be added for the so-called intangible values.

To emphasize the difficulties of appraising the intangible values in any way which will permit the application of such value to the determination of rates for transportation, the opponents of physical valuation cite what is now the familiar instance of two mythical roads between the same termini, the first with good alignment and easy

Mr. Lavis. grades following a valley, and the second forced into the mountains, having not only heavier grades and more curvature with consequently a higher cost of operation, but also more expensive construction. The value of the purely physical features of the former, of course, would be much less than those of the latter, but its actual value as a property would be greater. How then should the rates on the two roads be fixed? The fallacy of using this example as an argument against physical valuation as a basis for rate-making is in assuming that there would be two railroads built under such circumstances, with no other features than the two termini and the line between.

One has only to call to mind such examples of competing lines as those of the Denver and Rio Grande between Denver and Salt Lake, the Union Pacific between Cheyenne and Ogden, the Lackawanna and New York Central between New York and Buffalo, or many others, to realize that there are, on all roads of this nature, many other factors than the actual cost of operating through trains between the termini, which determine the through rates.

One would hardly suppose that at this late date any one believes that it is proposed to use only the value of the purely physical property of railroads as a basis for rate regulation, yet the *New York Sun,* a paper of national prominence and usually most ably edited, devoted a column of its editorial page* to a discussion intended to show that rate regulation, based on physical valuation alone, was an impossibility. In addition to citing the example given above, the following is put forward as the *reductio ad absurdum* of the argument for rate regulation based on physical valuation. It is said:

"Suppose there are two bridges over the Ohio, the *cost of the construction of each being the same,* one between Cincinnati and Newport and the other twenty miles below where there is nothing but a village on either shore. * * * On what basis would the proponents of physical valuation, as the determining value in rate making, adjust a toll charge on these respective bridges?"

The example is far-fetched, and in no way applicable to the question of the adjustment of rates on railroads, but inasmuch as it is seriously put forward from a responsible source, it seems worth while to consider it.

Assuming, as apparently the propounder does, that the proposition is uncomplicated by any questions of franchises, public rights in the land on which the bridge and its approaches are built, etc., then there is no question but that the owners of either bridge have a perfect right to charge what toll they please. On the other hand, suppose the permission of the War Department, or some other governing body, had to be obtained in order to build piers in the river, or even to build the bridge at all; the argument used in asking for this permission is that

* December 4th, 1910.

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the bridge is needed as a public convenience; or it is desired to occupy certain streets for the approaches, again is used the argument of public convenience, and so on. These privileges are granted on the tacit understanding, at least, that the public convenience is to be served, and the Courts rule that, in such cases, in consideration of the equity which the public has in the property by reason of the rights granted, a fair return on the value of the property, but no more, should be the basis for establishing the rates of toll. Would the *Sun* claim that the value of the rights and franchises given by the public in such a case, be included in the value of these bridges, and that a higher total income should be derived from one bridge than the other because the value of the streets on which the approaches had been built is greater in one case than the other; or that a greater income should be derived in one case than another because the cities furnish more people than the villages? Is there any particular reason, except for the slightly larger depreciation and cost of maintenance, and, bearing in mind the fact that both bridges cost the same, why, if there is ten times the traffic on one bridge than on the other, the toll should not be proportioned accordingly, to provide the same income on each?

If the *Sun* had imagined a bridge built by private individuals with their own money, between two villages, the inhabitants of which, at the time the bridge was built having been willing to grant almost any franchises or privileges in order to get the bridge, the villages in course of time growing to large cities, and the old bridge having been replaced by a heavier modern structure, the example might have been more nearly comparable to the railroad situation. In this case, the original toll, of say 10 cents a head, may have, in the early days, only barely returned a meager rate of interest on the investment, or even for some years resulted in a deficit. Would the *Sun* uphold the owners of the bridge if, since the villages have grown to cities, they still insisted on collecting the original toll, if it could be shown that a new bridge could be built and would be a paying investment with a toll of, say, 2 cents, except for the fact that the original bridge was built in the only location where it was practical to build a bridge at all? Or is it reasonable to say that the foresight and energy of the owners of the bridge, even though it may have been one of the principal factors in enabling the villages to grow into cities, entitles them to capitalize their enterprise on the basis of a 10-cent toll? It cannot be denied that the energy and foresight of the original builders should be recognized in fixing the rate of toll, but there is a limit to the value of this, and it is because of the feeling on the part of the general public that the capitalization of similar intangible values on the part of the railroads and other public service corporations is too large, which, whether true or not, has caused the present agitation against them. If the capitalization is reasonable, there must be some way to demonstrate

Mr. Lavis. the fact, and it seems as if a properly made physical valuation, with due allowance for the intangible values, is at least a step in the right direction.

The *Sun* states in its editorial that:

"The scheme of physical valuation, as a basis for rate making, is flatly rejected as unworkable by practically all the ablest railway authorities of the country, and that the only true measure of value is the earning capacity. To quote only one, namely: Dr. Emory R. Johnson, who is generally regarded as an authority and not by any means predisposed in favor of the public as against the railroads, we find that he states in his 'American Railway Transportation' that 'The earning capacity of the railroad cannot be equitably or logically made the sole criterion of value, because the rates, and hence the earnings, should depend to some extent, at least, upon the amount of capital justly entitled to profit.'"

It would seem to be self-evident that the earnings alone, either gross or net, are not necessarily an indication of the value of the road. Gross earnings are not, because, if a minimum proportion of them is used for maintenance and betterment, the value of the property will steadily decrease; whereas, if the opposite policy be followed, it will increase. On the same principle, the net earnings offer no criterion as to the manner in which the property has been kept up, and alone are, therefore, no measure of its true value.

As an example of the arguments used by some of the opponents of physical valuation, the following quotations are made from an article by Mr. Henry Fink, Chairman of the Board of the Norfolk and Western Railway.* Referring to the fluctuation in the costs of construction, he says:

"As the cost of materials and labor fluctuates * * * it follows that what may be a fair valuation of a railroad one year may not be so one or two years later. Hence, it would be necessary to make new valuations from time to time."

Further, in the same article, referring to a valuation based on the market value of bonds and stocks, he says:

"Unlike the physical valuation, this method has a rational basis. * * * It is true that prices of stock fluctuate—at times violently—but this difficulty can be overcome in a measure by using the average prices for long periods."

It is strange that it did not occur to so able a man as Mr. Fink that the value of the physical property might also be based on average prices for long periods; the cost of railroad construction and equipment as a whole does not fluctuate nearly so violently as the stock market.

The report on "The Basis of Unit Prices,"† by W. D. Pence, M. Am. Soc. C. E., the Engineer of the Wisconsin Railroad Commission, in

* *Railroad Age Gazette*, July 24th, 1908, p. 587.

† *Engineering News*, June 16th, 1910, p. 697.

connection with the Appleton Water-works case, is an excellent example of a fair and impartial study of this phase of the subject, and the conclusion of the Commission in this matter can only be regarded as reasonable by any one who is disposed to be at all fair-minded. He says: Mr. Lavis.

"If the standard by which the reasonableness of charges is to be determined should fluctuate with the market prices of material, labor and land, no schedule of rates could be established for any length of time, for, under the circumstances, a rate that would be reasonable to-day might be very unreasonable to-morrow. The principles of the law applicable to the subject certainly involve no such absurd consequences."

Another instance of an argument based on technicalities is found in the *Railway Age Gazette*.* In an editorial on Valuation and Rate Regulation, it is said:

"It has been supposed in the past that rate-making is an exercise of judgment. It seems to be assumed by many that after a valuation has been made it will be merely an exercise in mathematics. Suppose the value of a railway for state purposes is $50 000 000. Then, on this theory, all that will have to be done will be to multiply this amount by 6 per cent.—or whatever may be regarded as a fair return—and so adjust the rates as to enable the road to earn, say, $3 000 000 a year," but, the writer goes on to ask, "how are the specific rates to be fixed? A great majority of those who advocate valuation say that they should be based on the cost of the service. The proper method, then, would be to ascertain the exact cost of hauling each commodity and then base rates on these ascertained costs, making them just high enough to allow the road a fair return."

Then the article goes on to point out the difficulties of doing this, which of course we all know, and finally concludes that: "The theory of basing rates absolutely on the cost of service is unjust and impracticable." In the present state of the art this is probably true, but why is it necessary to change the present theory of rate-making because the rates are to be lowered or raised? If, for instance, it is shown that it is necessary to reduce the rates sufficiently so that the net earnings will be reduced, say, approximately 10%, is it beyond the capacity of the traffic officials of a railroad to adjust their rates accordingly?

In an editorial in another part of this same issue the *Gazette* advocates the raising of rates to meet higher prices of supplies and higher wages; it is surely as feasible to lower rates as it is to raise them, and, even though it were necessary to base rates on the cost of service, it does not seem as if that would be entirely impractical, inasmuch as it is the whole argument advanced for raising the commutation rates on the railroads entering New York City. Will the

* March 4th, 1910.

Mr. Lavis. *Gazette* say that the arguments put forward by these railroads are all wrong? Mr. Fink, in the article* already referred to, states:

"It cannot be said that * * * railroads make tariffs; they can only adjust them to varying conditions."

* * * * * * * * *

"Adjusting freight rates is practical work of men who have special training for it and large experience. They may not all be able to explain underlying principles, such as the value of service, but they have used this principle for years, and apply it intuitively in every case which comes before them."

Surely this body of men is equal to whatever adjustment may be necessary. Rates will probably never be arranged to suit every individual shipper; but if the people, as a whole, believe that the railroads are fairly capitalized on a reasonable basis of value, and the rates, in the aggregate, are adjusted so that unduly high profits are not made, individual complaints of injustice may easily be taken care of.

The most important considerations affecting the regulation of railroad rates arise in attempting to fix the amount which shall be considered a fair return on the investment. If a certain rate of interest is fixed as the maximum which may be earned, all incentive toward improvement or progress is removed. The effect of this would be, of course, to retard all development. Once a railroad was earning its legal rate of interest, there would be no necessity of cutting down grades, building larger locomotives to handle larger trains, investigating the economics of operation and location, in order to introduce the thousand and one economies which are being developed day by day, or for our railroad presidents to lie awake nights thinking how they are to save that million dollars a day for the benefit of the always ungrateful shipper. This objection against rate regulation, and incidentally against physical valuation, can undoubtedly be overcome. One proposal which has been made is somewhat along the lines on which it is proposed to finance the New York Subways, the profits to be divided between the railroads and the State, after a certain rate of interest had been earned. There is nothing novel about this, as several railroad charters have been granted with a provision that all earnings, over an amount necessary to provide a certain rate of interest, should be paid to the State. Another suggestion† is that the reasonable rate of return be fixed as a percentage of the gross income, irrespective of the amount of capital required to produce it. There are probably other ways in which this might be worked out and adjusted, and this phase of the subject surely does not present any insuperable objections.

That the railroads have little to fear, in regard to capitalization, from a properly made valuation, is shown by the results in the State

* *Railroad Age Gazette*, July 31st, 1908, p. 627.

† *Engineering-Contracting*, May 25th, 1910, p. 468

of Washington, where the valuation was undertaken solely for the purpose of fixing rates, the result being a determination of the market value of the three principal railroads of the State—the Northern Pacific, Great Northern, and Oregon Railroad and Navigation Company—at an amount considerably in excess of their capitalization.* It is true that rates were lowered in this case on some commodities, but it does not necessarily follow that every change of rates on the basis of valuation must be toward a lower scale. Railroad rates are low and have stayed low while the cost of everything else has been raised, and yet, while this fact is well known to the general public, they still believe that, in some way or another, the railroads are getting or have been getting more than their proper share of profits. Evidently there is something wrong somewhere, and it is not going to be set right by calling the public fools and ridiculing their presumption for meddling in any way with railroad affairs. Mr. F. W. Whitridge, the Receiver of the Third Avenue Railroad, of New York, while stating† that he had only just discovered that there was such a thing as valuation, at the same time held up the whole scheme to ridicule, though he admitted that: Mr. Lavis.

"The people of this country have, I think wisely, made up their minds, in consequence of great corporate abuses, that public service corporations should be subject to regulation, etc."

He nevertheless ridicules the efforts of the authorities, particularly their endeavors in the matter of valuation, with its "irreverence for facts." They seem, he says, "to be singing the song of the Banderlog who dreamed of

"'Something noble, grand, and good
Won by simply wishing we could.'"

Valuation, however, has gone far beyond the point where it can be considered a visionary scheme, or can be held up to ridicule; and it has been worked out far enough to show, at least, that there is a rational basis, on which a determination of values can be made, which will do justice to both sides; furthermore, the Supreme Court of the United States has not only ruled that valuation must necessarily be precedent to rate regulation, but has gone so far as to specify at least some of the elements which must be taken into account, and it may be worth while noting that, in spite of the author's criticisms of the Washington State Valuation, it is the only one, thus far, in which an attempt has been made to comply with the rules laid down by this Court. The results in Washington, however, indicate clearly the need of regulation of the railroads, as a whole, and not varied regulation by individual States of the parts of systems within the borders of each.

* *Railway Age Gazette*, March 4th, 1910, p. 437.

† *Electric Railway Journal*, January 15th, 1910, p. 110.

Mr. Lavis.

Arguments on either side can be prolonged indefinitely, and many goods reasons for and against physical valuation are advanced from time to time, just as they may be on any proposition. Some of the principal objections have been referred to here in an endeavor to show that they are not insuperable, the point which concerns us now is that to-day we are confronted with a fact and not a theory, and that fact is that the railroads are going to be regulated, and that their proper development is held back and general business is hampered by the feeling of uncertainty as to the outcome. Physical valuation is not a panacea for all evils, but a properly made valuation of the physical elements, with a due allowance for the intangible values, based possibly on some such method as that developed by the Washington State Commission or by Professor Adams in Michigan, is surely as good a way of breaking into the circle of argument as any that has been proposed thus far.

The equipment of freight trains with air brakes and safety couplers was practically forced on the railroads by the pressure of public opinion led by laymen, yet one will hardly find a railroad man now who will not admit that this is good practice, not only from the standpoint of safe operation, but from that of economy as well. The early attitude of the railroads in this matter is already being quoted by the advocates of valuation, and inasmuch as we have to admit, as we surely do, that a start is going to be made somewhere along the line of obtaining some more definite information in regard to the true relation of the value, capital, and profits, of railroad properties, than the mere statement by the railroads themselves that they are all that is good and fair, would it not be wise on their part to do all they can to have the start made properly rather than oppose it? Some of the most prominent and progressive railroad men of the country have already arrived at the point of believing and saying that regulation properly carried out may not be an unmixed evil, in fact, would probably be beneficial, but they still balk at valuation, without, however, suggesting any other means whereby the general public is to obtain the information on which to base an intelligent opinion as to how such regulation is to be carried out.

The speaker does not for a moment underestimate the difficulties incident to the determination of the intangible values, or forget the difference between the problem presented by the comparatively new lines in the State of Washington and a valuation of, say, the Pennsylvania Railroad or the New York Central. No one who gives any real thought to the problem pretends that the value of a railroad is the value of its purely physical property; but, because the matter of determining the intangible values is difficult and complicated, is it necessary that we should sit back and fold our hands and say "it can't be done"; that in the whole country there is no man or body of men,

or engineers, if you please, with brains and ability enough to solve the problem? As for cost, is it not worth $10 000 000, which is more than $40 per mile for all the railroads in the country, or about three times as much as the cost of the most careful appraisals yet made, to have the question put once and for all on a stable basis, satisfactory to all, if the problem be approached in a fair, broad-minded, common-sense way, by engineers big enough to command the respect of both sides? Aside from the question of rate regulation, is it not worth this much to the railroads of the country to be able actually to prove that the amounts at which they are capitalized are reasonable, as in the great majority of cases they probably are? Mr. Lavis.

There are one or two points which, it seems to the speaker, cannot be too strongly emphasized:

First, that valuations properly made may be the means whereby confidence may be restored, not only in the mind of the general public, but in that of the investor; but, in order to obtain this result, the railroads should urge, with all the power they possess, the necessity of having such valuations made by a body of men, some of whom, at least, should be engineers, big enough to entitle their opinions to the respect of both sides, and thoroughly qualified by training and experience for the work.

Second, that, as far as possible, regulation should be general or national, so as to avoid the complication of dividing all roads at the State lines, and of having different regulations in different States.

Third, that there need not necessarily be any relation between rate regulation and rate-making. Rate regulation can well be confined to rates in the aggregate, rate-making applies to the adjustment of individual rates, and must necessarily be the work of men well versed in all the varied elements which control it and the particular conditions affecting the business of each particular road. The speaker believes that valuations made in this way and with these objects in view will do no harm to the railroads, and will do much to restore confidence and give us the much needed peace and quietness to carry out necessary development.

Charles H. Higgins, Assoc. M. Am. Soc. C. E.—Mr. Riggs' able and timely paper is of great interest and worth to all concerned with the matter of values, whether of public service corporation property, or other property; and what engineer is not concerned with values? Mr. Higgins.

One cannot but wish that an index accompanied the paper, as its usefulness would be thereby greatly increased, particularly as, by its arrangement, such subjects as depreciation, non-physical values, etc., are treated of in many different portions of the paper.

The Wisdom of Having a Physical Valuation.—It is hard to understand how any thoughtful person can now doubt this, for we are in

Mr. Higgins. a period of regulation and taxation of public service corporations, and the only question is whether they shall be regulated and taxed with a full understanding of the investment involved, or by arbitrary methods, such as the 2 cents per mile passenger rate, which has been so popular in many States, under widely different conditions and irrespective of the cost of the service.

The time would seem to have arrived when the thoughtful public service corporation manager would welcome a fair valuation of the company's property, as protection against legislation conceived in ignorance of the capital invested.

Relation Between Railroads and Other Properties.—The relation between appraisals of railroad and of water, gas, and traction companies is very close, and the same general principles apply. In the former, however, it is complicated more often by the fact that the lines of a railroad extend through many States, with terminals in one or two, and, further, that the railroads have many subsidiary, controlled, or dependent companies, such as coal, lighterage, terminal, car, warehouse, contracting, elevator, stock yard, and supply companies, often owned, wholly or in part, by men in the railroad management. Agreements with these companies may greatly affect the non-physical values, as determined by the methods advocated in this paper, which may otherwise be sound.

Valuation of All Properties.—The author says that the valuation of all railroad properties in the country "would be of interest." It would be more; it would be of value infinitely greater than the cost. The mere presence of light prevents many vices, and this is as true in corporation practices as in the streets. It is in accord with Dr. Woodrow Wilson's "pitiless publicity"; and, which is, perhaps, more important, it is the basis, or should be, of all legislation concerning the regulations of these great highways.

One and Only One Fair Value.—Nothing in Mr. Riggs' paper is of more value than his insistence that there is one and only one fair value of the physical property of a railroad, no matter for what purpose it is to be used. How futile are the misdirected efforts of those who would have it otherwise, for, no matter what the purpose of the appraisal may be, who can foresee the use that may be made of it when it becomes public property?

Cost of Reproduction.—Cost of reproduction less depreciation seems to be the established method—that recognized by the Courts—for arriving at the value of the physical property. Cost, as the author contends, can only be an element in determining the present value, for the owner of a stone bridge has as much right to any appreciation in the value of masonry as the owner of land has in the increased value of his property; and, though the cost early in the life of the structure is usually near its value, it may lose that position. What

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relation exists between the value of the Pyramid of Cheops and its cost? Now, as then, our unit measure of value is changing. Cost is certainly of historic interest, but present value is the subject for present uses.

The points in favor of inspection to determine the physical condition of the object to be valued are convincing, where the structure may be readily inspected. Mortality tables mean little without a history of maintenance. With perfect maintenance there would be no physical depreciation.

Maintenance versus Depreciation.—Depreciation and maintenance are interdependent, so much so that some engineers have advocated dropping the term, "depreciation," and substituting "deferred maintenance." A little thought will make this clear. While this term would not apply in the case of a single rail or car, it is not illogical when applied to a system, built and renewed piecemeal and maintained at a certain standard of usefulness, that is, on all well-managed undertakings of magnitude, units are constantly being replaced, thus maintaining a standard of efficiency. This standard, on the entire system, is usually found to be between 70 and 90% of the cost of reproduction. Some items are even improved, and the cost is charged to the maintenance account, such as that referred to in the paper as "consolidation and adaptation" of roadbed, and only a few, such as steel rails, steadily and progressively become less useful, and even these have a bottom value, that of scrap steel. Nor are examples numerous where all the rails are laid at one time, and they are extremely rare where all are replaced at approximately the same time. When the rails on a street or section are renewed, the cost cannot properly be charged to capital account, except in so far as the new rails are of a more valuable type than the old ones; for, if this were done, there would be no limit to the capitalization as time goes on. Furthermore, the moment it is admitted that, by reason of a change in the art, we may have depreciation through obsolescence, we admit that through a change in art we may have appreciation through the opposite of obsolescence. This being the case, the use of "mortality tables" to determine present value is misleading, unless it is done with the full itemized accounts of maintenance, which are seldom, if ever, available. The author's position in regard to the need of inspection of each item is well taken.

Dead versus Live Properties.—These, perhaps, are not happy expressions, but they serve to emphasize a vital distinction which must be made in the valuation of properties. The difference may be as great as between a corpse and a man; here, also, the distinction is hard to define. We say the soul has departed, or the spark of life is extinguished, but these expressions do not contain a satisfactory scientific definition. So, as Mr. Riggs points out, the physical property of a going business may not be valued as so much junk, even if the non-physical values are to be determined separately.

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The Franchise a Contract.—The Courts hold a franchise to be a contract, something often forgotten, both by the public and by corporations. The speaker, however, understands this only to mean, even where the franchise is in perpetuity, that the property of the corporation cannot be taken for public use without just compensation. In a sense, then, there can be no such thing as a perpetual franchise. Using the word franchise with its restricted meaning, the unreasonableness of the rates may be measured by the value of the franchise.

Physical versus Non-Physical Values.—The following division has been made by the author between physical and non-physical property, for the purpose of valuation:

"That the Physical Value, or present value of the physical property, should fairly represent the actual capital invested in the property at the date of appraisal; that it should be made up of the sum of the various elements which constitute the cost of reproducing the property together with any appreciation which may have been added to any of them, less all depreciation.

"That the Non-Physical Value is the difference between the 'fair value' as defined by the Courts, or the reasonable value of the property as a business or producing property, and the physical value, or actual present worth; and that the only proper method for determining such values involves a study of income accounts.

"This Non-Physical Value may be: positive, or a value in excess of the physical property, or negative, or less than the physical value. In the case of a property having a negative intangible value, a deduction should be made from the physical value."

This division is convenient but arbitrary. It is the division of an engineer rather than of an economist; for these so-called non-physical values are like the breath of a man's life; without them, the physical value is like the discarded body. Again, the use of negative non-physical values, while convenient, may not be wholly logical. These remarks are not directed at Mr. Riggs, for he is careful to say that he is dealing only with active enterprises, and not with those which are inert, and the speaker realizes that he is not attempting primarily to build up a logical argument, but to formulate certain rules to overcome practical difficulties met by all who have attempted valuation work. As many who have not given this matter much thought are apt to be misled by the distinction made between physical and non-physical values, they should bear in mind that the line between them is like the equator, an imaginary one.

Water.—"The water is as much a part of the cost of putting that line there as the rails," remarked a corporation official, of admirable character and wide experience, pointing to a trolley line from the window of a Pullman car; and, bearing in mind what he meant by "water," this is undoubtedly so. The cost of promoting the enterprise, the discount on the hazard, the loss of interest during its infancy, the

labor of building up the undertaking—these are all real elements of cost, and may remain in the property as value, but, like all other items of cost, they have their reasonable limits, which, in each individual case, can be determined within narrow bounds. Mr. Higgins.

Purpose of a Valuation.—As Mr. Riggs points out, there are four reasons for a valuation: Taxation, rate-making, purchase, and control of the issue of securities, one of which is usually the primary cause for the valuation being made; and he argues that there can be but one "fair value" of the physical property, whichever of these reasons may prompt the appraisal. This is fundamental, for "fair value" is used in the sense of true value, which, to the writer, seems to be a more apt expression. It is rather surprising that it does not appear in the paper. Its use, of course, is old; in the Constitution of New Jersey, 1875, we find: "Property shall be assessed for taxes under general laws, and by uniform rules, according to the true value." Each of the three matters, taxes, rates, and authorized capitalization, are interdependent and, in the long run, cannot be considered separately. This can be emphasized by a *reductio ad absurdum:* Modern civilization is so dependent on transportation by rail that unquestionably all taxes could be raised by assessment on the railroads, if these roads were allowed to fix their rates and were protected in the collection of them; but how would this method differ from that of the Romans, of farming out the collection of taxes? Not materially, and no one advocates a return to that method. This is absurd, but it serves to emphasize the relation between taxes and rates. Taxes can only come from the rates.

Overhead Charges versus Unit Values.—There is much in various parts of this paper concerning overhead charges, but very little about the items considered in determining the unit values or unit prices used; and does not the latter greatly affect the former? For example, in discussing the Michigan appraisal, the author says:

"For many items, such as clearing, grubbing, earthwork, masonry, etc., the price was fixed by agreement during the discussion at a figure which represented the fair average cost of this particular item during the 5-year period preceding the appraisal."

The "fair average cost" under what conditions? This word "cost" is understood by different men in as many different ways as the word value. Mr. Riggs very clearly gives the items included in "fair value" as finally arrived at by him, but it would seem to be as important to define "fair cost" as used in arriving at the unit prices, for otherwise the chain has a weak link.

What may be considered a fair cost per unit of measure for a particular item differs greatly: First, with the point of view and breadth of horizon of the man stating such cost; and second, with the methods of letting contracts and accounting with which he may be

Mr. Higgins. familiar, as applied to such items of work. Because of the first, a fair average unit cost may mean one thing to a contractor, another to a division engineer, still another to a chief engineer, and a fourth to a manager or consulting engineer; and because of the second, the understanding of the term may differ among men of the same class. All of this quite aside from what may be termed the personal equation of the individual. Thus the subject of overhead charges can only be discussed profitably in the light of knowledge concerning what has already been included in fixing the unit prices used. For example, the element of hazard common to all construction, but differing in degree on different classes of work, may be included in the unit cost used, or it may be added as a percentage to resulting sums, but it cannot rightly be included twice. This is equally true of other elements of cost of a similar character.

The foregoing is pertinent, for any valuation will probably be attacked in the Courts, and the unit values will be one of the most tempting points for assault, for the very reason that this wide difference of understanding in regard to cost, and particularly in regard to unit costs, exists. This same difference of understanding is usually the reason for the wide difference in unit costs testified to by able engineers and, consequently, for the distrust often felt for such testimony. The methods followed in taking expert testimony usually work to make "confusion worse confounded." The judge or layman, hearing two engineers testify to widely different unit prices as a fair average cost for certain work, forms a low opinion of their judgment, or worse, whereas the real difficulty may, and usually does, lie in a different understanding of the meaning of the term "cost," or "unit cost." To the speaker, this seems to be the weakest point in an admirable paper.

Paving.—Whether the value of the paving between and for a space outside of the tracks is an element of value in a street-car line, or whether the cost incidental to the construction and maintenance is in the nature of a tax, is a much disputed point in all valuations of street-railway properties, and an important one, for it may amount to $15 000, or more, per mile. It is interesting to remember that the custom of requiring street-railway companies to maintain the pavement between its rails and for a space of about 2 ft. outside of them, which has become almost universal, developed during the use of horses to draw the cars, the animals causing great wear on that portion of the street. This question of values is a difficult one. It would seem that the most tenable position is that: If the fee to the pavement is not in the company, and if the rule concerning cost of reproduction less depreciation is to be followed, the cost of taking up and relaying the pavement is an element of value in the physical worth of the track, for it would be impossible to reproduce the track without incurring the cost of such work.

S. D. Newton, Assoc. M. Am. Soc. C. E. (by letter).—The general scope of this paper is admirable. The author's views and definitions are unusually sound, clear, and forcibly expressed. To one minor detail, however, the writer is unable to subscribe. Referring to "the physical property element of value," he states that: Mr. Newton.

"This consisted of those things which are visible and tangible, capable of being inventoried, their cost of reproduction determined, their depreciation measured, and without which the property would be unable to produce the commodity on the sale of which income depends."

Take the case of an industrial spur for some minor industry along a line of railroad. It is often a question in the minds of the management whether or not the car-load business done by such an enterprise is sufficient in quantity to warrant the expense of a spur track. There are probably other facilities in the neighborhood which could be used to take care of this business at the expense of some inconvenience; in a large proportion of cases, the railroad will handle the business anyway, and the spur can in no sense be called a necessity. Still, it is visible, tangible, and capable of being inventoried, and should be included in an inventory of the property the same as any track or section of track belonging to the Company. This may also be said of an extra settling basin or filter bed in the case of a water-works plant. If such basin or bed were not in existence, and a leak should occur in the original plant, the business of supplying water to its customers could, in all probability, be carried along in some manner until the break could be repaired; nevertheless, such a tank or bed is desirable, and its value should most certainly be included in an inventory.

Take the extreme case of a piece of machinery which is utterly broken down or so far out of date as to be entirely worthless for the purposes for which it was designed. Yet such machinery has, at least, a scrap value, and as such it should be included in the inventory as part of the tangible assets of the concern at the date in question.

Of course, in many instances, certain interests endeavor to have inventoried items which should either be omitted altogether or included at a much reduced valuation from that sought to be placed on them, and, in such cases, the very best judgment of the appraising engineer must be called into play in order that injustice may not be done to either party; but to say, as Mr. Riggs' definition virtually does, that nothing should be inventoried which can, either with or without inconvenience, be dispensed with, is absurd, and the writer does not believe that such is the meaning the author intended to convey. Probably, if the word "economically" were inserted in the definition, it would more nearly represent the proper idea.

William V. Polleys, M. Am. Soc. C. E. (by letter).—In his very thorough and painstaking paper Mr. Riggs states that it is confined Mr. Polleys.

Mr. Polleys. to a discussion of methods for arriving at a correct figure of cost, and disclaims any intention of considering the propriety of using said figure when reached.

Inasmuch, however, as he devotes the next eight or ten pages to a dissertation on law, political economy, rate-making, finance, and advice to railroad employees, with a word of encouragement to the good, and firm reproof to the bad ones, it is fair to assume that he intends this disclaimer in a Pickwickian sense, and that the real intent of the paper is to show that the physical valuation of property is, with certain determinative, corrective factors, a proper standard for gauging taxation, bond issues, and kindred evils.

Is it not a fact, however, that taxation is based on a much more intangible structure, and that the net earnings must necessarily have more to do with it than the physical valuation of the property—whether it be that of a wicked public service corporation, or that of an honest haymaker—rather on what their property can produce, than on what it would cost to produce the property? Is it not rather a battle of business acumen between the taxer and taxee, a battle which, among other things, is regulated more or less by the fact that an extreme in either direction will bring disaster to one or both, followed by the inevitable reaction and readjustment.

Take, for instance, an extreme case: A manufactory is erected on comparatively worthless ground. A million dollars or more is invested in a plant, with the result that surrounding real estate values go up with a bound. Supposing that the manufacturer has not made any previous arrangements for immunity, and the assessors are both acute and honest, the property will be taxed for a large figure, which tax, if the factory is making money, will be paid, with more or less grumbling, up to the economical breaking point. Suppose that, owing to a sudden permanent change in business conditions, it becomes impossible to operate this plant, and it is abandoned. A corps of experts may be thrown into the mill, before the last employee has left the building, and may carefully scrutinize and caliper the machinery, count the bricks in the wall, tap the stay-bolts in the boilers, and bore into the furniture to see whether it is solid or veneer, and when they are through and their figures are all in, they have not arrived at anything that is of the slightest use as a basis for a bond issue or taxation, and very little that would be of use for sale. In such an extreme but by no means unheard of case physical value bears no relation to real value.

This is not to say that a physical valuation is without worth, and even great worth in some cases; it is merely offered as an opinion that the physical value is in many and probably most instances, a very treacherous guide to the real value—a far poorer guide, as a general rule, than the accounting department; a minor quantity, in fact.

It seems doubtful whether there is a scientific way of arriving at the

true value of a going property by the physical-valuation route. There is too large a percentage of values which, being intangible, are matters of judgment. At best, the determination of value must be that of opinion, and the worth of that opinion hinges principally on the practical qualifications and disinterestedness of the person who gives it. Mr. Polleys.

Unfortunately, or fortunately, as the point of view may be, the disinterested person is not apt to be qualified, nor the qualified person to be disinterested, and it seems extremely probable to the writer that, while weapons may be changed and excuses vary, the tax war will be waged as of yore, and the fool and his money will continue along diverging paths until something more ingenious than physical valuation is invented, however well the valuation may be made.

C. P. Howard, M. Am. Soc. C. E. (by letter).—While there may be no material differences of opinion as to the principles on which a physical valuation should depend, such a detailed description of organization and methods as that presented by the author should be of great service to others undertaking similar investigations. Mr. Howard.

It may not be amiss, however, to mention certain features affecting the non-tangible values which should be more fully considered in any general discussion of the subject.

The author calls attention to one or more particulars in which the methods of the Michigan appraisal may "fail as a method of determining a value for use as a basis of rate-making." Later, after quoting various court decisions, he dismisses this phase of the subject with the words: "In view of these dicta, it is needless to argue whether a rate of 6% or 10%, or 15%, or more, be reasonable."

A value for purposes of rate-making might more properly be called a "permissible value." The writer holds no brief for the corporations, and would not like to fall under the imputation of being "apparently incited by, either the direct interest of corporations, * * * or an effort to confuse the subject of valuations," but will venture the following, which, while it does not exactly represent any particular case, it is hoped may be recognized as an illustration drawn from life.

A, B, C, and their associates, being familiar with a certain territory, its resources, transportation facilities, and growing development, believe that the time has come to build another railroad through their State or States. They have made careful estimates of the amount of tonnage that may be expected from the development of its mines, timber, farms, etc., and conclude as follows:

First.—The road, completed along the most approved lines, will cost, with equipment, $50 000 000.

Second.—It will take five years to construct and equip the road and put it in fair running order.

Third.—The traffic, when fully developed according to their hopes and expectations, will eventually afford at usual tariffs a handsome

Mr. Howard. profit, say, from 8 to 12% per annum on the capital invested. This condition, they believe, in all human probability will be attained in from 5 to 10 years after completion.

Fourth.—That half the traffic anticipated will pay 5% on the investment.

Fifth.—They are obliged to admit (though the chances of this are so remote as to be in their opinion negligible) that, due to unforeseen causes, obstruction, competition, etc., there is a possibility that, as has so often happened in the past, the enterprise may prove a financial failure, or that the period of prosperity may be postponed so far into the future as to amount to practically the same thing.

Here is a bold undertaking; but were it $5 000 000 instead of $50 000 000, the conditions would be essentially the same. Nevertheless, they have the courage of their convictions and go ahead.

Now, with all the risks and uncertainties attending an enterprise of this sort, if the ultimate profits were limited in advance to 5 or 6% on the capital invested, less depreciation, who but the Government itself could afford to build a railroad?

Evidently, when an existing railroad makes small additions from time to time to extend or take care of its business, the risk is not so great. Such extensions will continue more or less under any limitations.

For rate-making, it is evident that an appraisal based on earnings will utterly fail of its purpose if made during the lean years immediately following construction. If made some years later, when the property has begun to pay, the risk and necessary financial loss of the lean years should be remembered, as any one building a road in the future will necessarily have the same problems to meet, together with the expenses of interest, depreciation, loss from operation, etc., both during the construction and the lean years following, all of which must properly be considered a part of the real cost of constructing and developing a property.

Mr. Willoughby. J. E. Willoughby, M. Am. Soc. C. E. (by letter).—The determination of the cost of reproducing the property of any steam railway involves, together with other items, an estimate of the present cost of:

First.—The acquirement of the right of way, to the extent, in the form, and on the location of that held in connection with the railway to be reproduced;

Second.—The construction thereon of the roadway, to the form and dimensions, and of the materials which the roadway to be reproduced exhibits; and

Third.—The seasoning and adaptation of the roadway to the state of perfection which the roadway to be reproduced exhibits at the time the estimate of cost of reproduction is made.

The first conception, for fixing the cost of the several items, is to consider the railway to be reproduced as being non-existent at the time the estimate is made, but having the environment which then exists along the operated railway, although that environment may be largely of the railway's own creating. The cost of the right of way is to be fixed as ungraded and unimproved property attached and forming a part of the adjoining improved property, which adjoining property will be entitled to receive, in addition to the market value of the land taken, all consequential damages due to the taking off of the right of way in the form and location that the land has actually been taken, and for the purpose of railway construction and operation. This adjoining property is to give credit on the consequential damages for the incidental benefits which it derives, if any, from the construction and operation of the railway. Mr. Willoughby.

In fixing these values, the drift of public sentiment—the bias of juries of view and of trial juries—at the time the estimate of cost of reproduction is made must be considered, since that sentiment may affect enormously the cost of the right of way. The amount to be paid for a right of way is in the end that which a condemnation court will award. The question as to whether or not the right of way was originally donated can no more enter into the determination of the cost of reproduction, for the purpose of lessening the estimate of cost of acquiring the right of way, than the fact that donations of lands or bonds (or of convict labor and slave labor, as in the South prior to 1860) made by governmental authority or private enterprise, at the time of the original construction, can be used to reduce the reproduction cost of the excavation made in the formation of the roadway.

No rule as to the sale of property for commercial purposes in the vicinity of an operated railway can be rightfully adapted as covering the line as a whole. While the cost of right of way through farm or timber lands bears a general relation to the value of those for agriculture purposes, where improvements thereon bear but a small proportion to their total value, this relation is wholly wanting in the cost of a right of way through a village or city or at any point where the improvements on the property bear a large proportion to its total value. The relation is also wanting where a right of way is obtained through agricultural lands devoted to special purposes, like that of country homes for the rich. It is also wanting where the right of way is taken out of the narrow river lands in the Appalachian Mountains, where the total value of the whole farm is dependent on the small acreage of flat land along the river bank. The general rule of prefixing a constant to the current selling price of lands, in order to determine the estimated cost of right of way, should be limited to agricultural and timber lands, and to those which, owing to their

Mr. Willoughby. extent, the carving out of the right of way does not wholly destroy for the continuation of agricultural and timber operations.

For villages and cities, and for lands devoted or adapted to special purposes, an accurate estimate of the cost of reproduction of the right of way can be determined only by a specific investigation of the conditions in each community. While it is difficult to conceive all the activities and sentiments which have growth in, from, and of railway operation, as being in existence without the railway, it is only through such an assumption that one can estimate correctly the make-up of the items of cost of reproducing a railway as such railway may now exist. To assume that the railway, not existent for the purpose of estimating the cost of reproduction, will now receive the donations of land and moneys that were made half a century ago, is merely going back to a determination of what the road has actually cost; and that is contrary to the intent of the theory of the cost of reproduction. The conception of a parallel line is not correct, for it imposes thereby a further burden on properties which have already contributed to the public good, probably to an extreme extent, and gives an abnormal cost for right of way, as shown when a railway seeks to enlarge its terminals in a crowded community, or to find a new entrance into a populous city.

So, too, in estimating on the formation of the roadway, one must consider the roadway to be reproduced as being obliterated—all cuts and borrow-pits refilled, and all embankments and spoil banks removed from the right of way—but all other lines of transportation, except the railway to be reproduced, must be considered as being in existence as they actually are at the date the estimate of cost of reproduction is made, and that such other lines of transportation are available for bringing in machinery, tools, teams, materials, and supplies for the construction of the railway to be reproduced. It is only by such an assumption that the benefit of the improved means and methods of construction now prevalent can be obtained; but it is not permissible to estimate for the construction of a railway with different grades, alignment, roadbed, widths, or with different materials than that of the railway to be reproduced merely because such construction at this day might be actually cheaper or better than to construct it in exact duplication. For example, if the rock cuts on the roadway to be reproduced be only 18 ft. wide with $\frac{1}{4}:1$ slopes, one must not figure on the greater economy of steam-shovel excavation, because the steam shovel cannot be worked in cuts of that width; nor can the spoil from such cuts be carried long distances to eliminate a possible solid-rock borrow originally made elsewhere, because long hauls are practicable in steam-shovel work, but wanting in excavations where the mule is the transportation force. So, too, it is not permissible to estimate on reinforced concrete bridges to take the place of more costly cut-stone

arches, if cut-stone arches are the structures that have been actually built. The idea of cost of reproduction is not synonymous with the idea of the cost of building a railway capable of serving the same transportation purpose. If all our railways were to be built anew, in the light of our present knowledge, and with our present traffic offerings and financial resources, vast changes would be made in the character of construction. The physical fact of existing construction prevents a theoretical substitution of what is the best construction for any community, together with its costs for the construction which was actually made years ago. Mr. Willoughby.

In the event that an estimate of reproduction costs be made for a State as a whole, or for a great railway system as a whole, the conception of reproduction is modified so that the construction may take the form of progressive construction, the principal lines being built first and the less important lines afterward. This method will require the estimate for interest during the construction period to be greater.

The money cost of the seasoning and the adaptation of the roadway to such a condition as will permit heavy trains to be run at high speeds, is great, but the amount is not readily ascertained. An estimate of cost of reproduction, to be true, must consider this item; and probably the more usual method of ascertaining it is to assume it to be an amount in some proportion to the cost of the excavation. This proportion will vary with the character of the material through which and of which the cuts and fills are made, and with the methods of construction necessarily adopted. There are many railways on which this cost will exceed 25% of the total cost of excavation.

After the estimate has been made, including the item for seasoning and adaptation, there should be added a contingent fund to cover the omitted work, consisting of small borrow-pits and ditches, undetermined foundations, unexpected conditions encountered, unavoidable "force account" work, minor changes of streams and highways, damages to adjoining lands due to the methods of construction and to diversion of water, etc. This item will not exceed 5% of the cost of the roadway if the estimate be accurately made.

The more convenient form into which an estimate of cost of reproduction of a steam railway is to be put is to follow the sub-accounts, as prescribed by the Interstate Commerce Commission for Expenditure for Road. Each item given in that accounting has a place in the estimate. These comments are confined to the items covering the roadway, namely, Right of Way and Station Grounds, Grading, Tunneling, Bridges, Trestles, and Culverts.

HENRY C. ADAMS, ESQ.* (by letter).—To the writer this paper seems to be the most complete and comprehensive discussion of the general question of valuation of property invested in public service industries that has come under his notice. It is especially important in Mr. Adams.

* Professor of Political Economy and Finance, University of Michigan.

Mr. Adams. that it is a summary of the discussion on this most difficult subject during the past ten years, and the writer thoroughly agrees with the general conclusions reached by Mr. Riggs.

There is one point, however, which might possibly have been developed more completely, and that is, the treatment of discounts, which presents itself from time to time in the general discussion. Mr. Riggs quotes with approval the following:

"If a company can market its 50-year, 4 per cent. bonds at 90 per cent. of par, it means that the company's credit is on a 4½ per cent. basis; that it could market a like security paying 4½ per cent. at par."

This is, of course, correct as far as the mathematics of the proposition is concerned, but it seems to overlook that peculiar psychology of the market which enables a corporation to secure a larger amount of actual cash for a given interest annuity when bonds are sold at a discount than when they are sold at par.

Aside, however, from the accuracy of the above quotation and of Mr. Riggs' apparent acceptance of it as the final word on discounts, one may ask if it recognizes all the elements necessarily involved in a discussion of the problems raised by discount financiering. From the literature of the subject one may read the following claims: Discount is a measure of the risk involved in a new enterprise; discount is a market adjustment that reflects the current value of money; discount is a sacrifice of principal for a slightly reduced interest annuity; discount is a dividend declared before the dividend is earned; and many cases are cited in which a discount is merely a promoter's fee for services rendered.

The writer does not care to discuss at this time these various points of view from which discounts may be regarded. They are mentioned merely to suggest that the subject is not as simple as some writers seem to think. Any valuation of public service industries, from whatever point of view it may be regarded, must, from the nature of the case, touch the problem of fundamental equities; and one of the elements of this problem which has not as yet been fully analyzed is this element of discounts. From the point of view of taxation such an analysis is not perhaps essential; but if the valuation is to be used as a basis of determining reasonable rates, or as a measure of reasonable capitalization, it seems to be essential.

The writer is sure this discussion will not be construed as in any sense a criticism on Mr. Riggs' paper; it is rather a suggestion of an unwritten chapter in the literature of valuation. The American Society of Civil Engineers is to be congratulated in securing from one of its members so complete and satisfactory a discussion of the principles and methods for the valuation of public service corporation property.

Carl C. Witt, M. Am. Soc. C. E. (by letter).—The appraisal of the railway property in Michigan was a wonderful performance in a great many ways, not the least of which was the thoroughness of the work, considering the short time available, and the writer desires to express his appreciation of this paper, as it is a valuable addition to the meager literature on this subject. Mr. Witt.

More recent appraisals, made by States traversed by the same railway systems as those involved in the Michigan appraisal, have been made with a freedom from opposition by railway companies due to the educational effect of this pioneer work. Particularly is this true of the recently completed appraisal, by the Board of Railroad Commissioners of South Dakota, of the physical property of the railways in that State, of which work the writer was the Engineer in charge. No opposition was met, in fact, some of the railway companies had established regular departments for furnishing inventories and appraisals, had completed the necessary field work in South Dakota before the inventory had been requested by the State, and were able to furnish a very complete appraisal in a short time after the request for it was made.

This appraisal was made in compliance with an act of the Legislature of 1907, which required the Board of Railroad Commissioners to ascertain the true cash value of all the property of every railroad company in South Dakota used in the operation and maintenance of their respective roads. No attention was paid to the purpose of the appraisal, but one of the first uses made of the information thus secured was in the litigation following the passage of an act by the Legislature of 1909, prescribing a maximum passenger fare of 2 cents per mile on all railroads operating within the State. In connection with a rate case of this kind, some questions have been raised regarding proper bases for land values, the use of an item for adaptation and solidification as an element of physical value, the value of the intangible assets, etc.

The lands of all railway companies were appraised at a cost to reproduce or re-purchase at the time of appraisal, regardless of the original cost of the property. The sales method was used for determining the market value of adjoining property. There has been a very large land movement in South Dakota in the last five years, and as most of the country is prairie, with similar soil over large areas, it was not difficult to determine the average market value of the land for farm purposes, at the date of the appraisal, and the gradual trend of values for five years previous to that date. An average multiple of 250% was used to arrive at the cost of reproducing or purchasing the right of way. This multiple was based on investigations made of recent right-of-way purchases, and inasmuch as there are no large terminals in South Dakota, the same average multiple was used

Mr. Witt. throughout the State for both town and farm property, investigation showing that town property could be secured for slightly less and farm property for slightly more than the average multiple used.

In each supplemental appraisal the land values will be corrected to correspond to the changes in surrounding values, as the railway company is entitled to any increase, due to natural causes, based on the cost to reproduce at the time of appraisal. This is a well-established theory, as shown by Mr. Riggs.

No allowance was made for the item commonly known as "adaptation and solidification," except in the item of contingencies and in the consideration of the present value of the ballast. In some recent appraisals, large sums, based on a percentage of the cost of grading, have been allowed for this item. While there is no question that large sums of money are expended in maintaining a safe track on a new bank, and that this expense gradually diminishes as the roadbed becomes solid, due to the pounding of the trains and the action of the elements, this expense is, and properly so, charged to maintenance, and is paid for out of the operating revenues. Now, in the trial of a rate case, exhibits showing the operating expenses, including maintenance charges, are introduced, and to include this same item in the appraisal of the physical property leads to a duplication, for if the passenger or shipper pays for this maintenance charge, it should not be counted as an item of physical value as a basis for determining what is a reasonable rate.

The case is similar to that of a locomotive; when new, it is kept in the vicinity of the shops, because trouble from lack of proper adjustment and weak parts is likely to develop, and the maintenance charges may be much higher than a few months later when the machine has "found itself" and, as an operating machine, is more efficient than when new. However, no one will insist that it has an added physical value in dollars and cents, or that the excess cost of repairs and maintenance during its early life should be added to its cost of reproduction now; in fact, it is a second-hand machine, and the maintenance charges must be paid for out of its use.

Generally, when a roadbed is turned over to the operating department by the construction department, it is in good line and surface, and if an appraisal were made at that time its condition would be 100%; but as soon as it is placed under traffic, it begins to depreciate, as shown by the fact that it requires constant attention to keep it up. If the roadbed is cross-sectioned at each station and actual quantities calculated from cross-section notes, there would be no depreciation, but if the grading quantities are calculated from profiles of the line, as constructed some time previously, and for a standard width of sub-grade, with a percentage added for shrinkage, and allowance made where banks have been widened, etc., it will probably be found

to exceed the actual measured quantities, because the action of the elements in washing the slopes, the wearing of the shoulders of the embankment due to foot traffic, etc., will show some depreciation in quantities. It is common practice to carry the item for grading over to the present-value column at 100%, or, with no depreciation. This practice, together with the present condition of the ballast due to maintenance, and that part of contingencies which covers washing of slopes, filling of ditches, sink holes, etc., certainly takes care of all adaptation and solidification which should enter into a valuation of physical property. Mr. Witt.

No appraisal was made of the intangible assets. A great many arguments have been advanced for and against such an appraisal, and in South Dakota it was held that the earning ability of any corporation due to its franchise, strategic location, efficient organization, going-concern value, etc., while perhaps an element of value to be considered in a transfer of the property or if assessed on an income basis, should not enter into a valuation which would be used for determining a just and reasonable return on the investment, because the greater the earning power the greater would be the return, and that this condition would produce a never-ending increase in returns; whereas, when the returns reach a point at which they will not only pay a fair dividend on the investment, but take care of any depreciation in the physical condition of the property and make all needed improvements in roadbed, buildings, and equipment, demanded by the traveling public, shippers, increased traffic, or natural causes, they should be kept to that point. There are several hundred miles of railway in South Dakota which have been built out of the surplus earnings of the parent corporation—in other words, with money supplied by the traveling and shipping public, but which are owned by the railways and on which they may earn another surplus for constructing more extensions, etc., etc.

The original South Dakota appraisal, as of June 30th, 1908, on forms similar to those used in Minnesota, has been supplemented by yearly appraisals corrected for all additions and deductions made during the fiscal year. For this purpose a new set of forms* was prepared, with the various items classed in accordance with the "Classification of Expenditures for Road and Equipment," as prescribed by the Interstate Commerce Commission, and arranged so as to facilitate showing the yearly changes.

R. A. Thompson, Assoc. M. Am. Soc. C. E. (by letter).—This paper is considered by the writer to be the most complete treatise ever written on the valuation of public service corporation property, and the author deserves the sincere thanks of the entire Engineering Profession and all others interested in this most important question. Mr. Thompson.

*For convenient reference, a set of these forms is filed in the Library of the Society.

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Its presentation is most timely, in view of the agitation, particularly on the subject of railroad valuation, which is now engaging the attention of Congress and the law-making bodies of the several States, as it contains much valuable information relative to decisions of Courts, in addition to clear and concise expositions of the methods in vogue for the appraisement of corporate property, etc.

It is a fact—rapidly coming to be recognized by legislative and judicial bodies—that the prescription and regulation of tolls, charges, and assessments against public corporations cannot be made systematic and intelligent unless there is provided some estimate of the value of the property involved, based on the cost of its replacement or reproduction. Particularly is this true of railroads; and such regulation of the affairs of these corporations as has heretofore been essayed by State and National commissions, has generally been on illogical bases, unsatisfactory alike to the proponents and the companies. Results have been had, it is true, after a fashion, but there have been endless disputes and litigation, with the prime questions involved no nearer solution than before. One has but to contemplate the varied and often antagonistic legislation promulgated by the several States, relating to corporation management, and the many rulings and decisions of the different courts and commissions on the subject of regulation, assessment, and adjudication of corporate rates, revenues, taxes, and tolls, to become convinced of the complicated and tangled condition of the situation, and to realize the necessity for the early establishment of some logical basis on which to establish the fabric of corporate control.

While it is not maintained that an appraisement of the physical property of public service corporations will be the panacea for all such ills, the writer firmly believes with the author that such appraisal, as a beginning, is absolutely necessary, and when effected on some fair and reasonable basis, will contribute largely to the successful solution of many of these intricate problems.

With the estimate of the physical value of a property before it—which represents money actually invested, together with such accruals to costs as it may be determined that the owner is reasonably entitled to have considered—any Court, tribunal, or commission is in a better position to mete out impartial justice, whether it be the regulation of a rate, the assessment of a tax, or the imposition of a fine.

Although the author's experience in valuing corporate property has been principally in connection with the Michigan appraisal of railroads, and to him is largely due the credit for devising methods for, and carrying forward to successful completion, this thorough and most excellent work, it is refreshing to note his inclination to give credit to the work of others along the same line in other States, which, it is to be regretted, has not always been the case with writers

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on this subject. There is no doubt that the work of the Michigan and Wisconsin Boards of Appraisal—conducted under the advice and direction of some of the most eminent and talented engineers and economists in the United States, and practically without regard to expense—is the most complete and perfect of its kind heretofore attempted, yet there are many features in regard to the organization and execution of its details about which there may be an honest difference of opinion, as viewed by those who have been similarly employed.

It is but natural—as suggested by the author—to find the "individual" character of the appraiser (which has been moulded by his environment, training, and former service) reflected in his opinion, and this would be most probable in the organization for, and carrying on the work of, appraising a railroad property, which involves consideration of practically every phase of engineering and economics. The judgment of any man is essentially warped along the lines of his experience, and he is necessarily biased and prejudiced in favor of or against certain practice. As a consequence, therefore, it is not reasonable to suppose that any one man, or set of men, can formulate a system for valuing corporate property which will be perfect in all its details, and be free from objection and criticism.

The writer was employed for a number of years as Engineer for the Railroad Commission of Texas, and had charge of the valuation of railroad property under the Railroad Stock and Bond Law of that State. A paper on the methods used by this Commission was prepared by him and published by the Society.* This Stock and Bond Law was enacted in 1893, and the railroads then existing were valued in 1894 and 1895. The average value of 8 860 miles was $15 844 per mile. This valuation was made by Charles Corner, M. Am. Soc. C. E., now Resident Engineer of the Rhodesia Railways, South Africa, and Mr. H. J. Simmons, now General Manager of the El Paso and Southwestern Railway System. The actual cost of this work is not available, but is estimated at about $2 per mile. The engineers making the appraisal secured maps, profiles, and all available information from the offices of the railroad companies, including all the construction records and estimates of quantities which were preserved. Appraisal was made only after one of the engineers had made a personal examination on the ground, accompanied by assistants to aid in measuring structures and estimating quantities.

All valuations made since 1895 have been of new railroads making application for issuance of securities, and in all cases the deeds for right of way and depot grounds, the contracts for construction, the actual quantities of construction of all kinds, the plans and specifications for all structures and construction, and all other information

* *Transactions*, Am. Soc. C. E., Vol. LII, p. 328.

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Writers on railroad valuation have generally been inclined to discredit the work of the Texas Railroad Commission and the system of appraisal used by it. One writer, of more or less prominence, has referred to it as the "cheap" method. While it may be true that other appraisals have been more expensive, it is a fact that those of the Texas Commission have served their purpose well, and the railroads, as a rule, have made little complaint. As a matter of fact, it is highly probable that the valuations of railroad property made by the Texas Commission have been of greater utility, as far as the public is concerned, than those of all other States combined, and, at the same time, no injustice has been done the railroads.

It appears that those who have interested themselves in investigating the Texas method of railroad valuation—including the author—have failed to construe the real meaning and intention of the Stock and Bond Law. Apparently, it was passed for the purpose of limiting railroad indebtedness—and is referred to by Mr. Riggs as serving only this purpose—but while its effect has been to accomplish this most successfully, its enactment carried with it a deeper significance.

This law was passed at the same time as the General Railroad Commission act of the State, which gave to this Board absolute control over all freight rates and tariffs, and also other powers not possessed at that time by any other State commission. The decisions of the highest Courts at that time laid stress on the right of carriers to maintain rates which would afford a reasonable return on stocks and bonds outstanding. Hence, to delegate the regulation of rates to any tribunal by any law which did not carry with it also the right to supervise and restrict mortgage indebtedness to some reasonable extent, appealed to the legislators as being essentially ineffective. The effect of the law has been to reduce steadily the average outstanding stocks and bonds of the railroad companies of the State from an average of $40 802 per mile in 1894 to $31 910 in 1909—and this, too, in the face of a recognized increase in the physical value of the properties—thus depriving the railroads of one of their most potent weapons of offense when contending against the Commission's orders. It is a matter of common knowledge that the indebtedness per mile of railroads of other States has increased greatly during this period. It is also a fact that the railroads of Texas have, except in rare instances, contended that injustice has been done them in the enforcement of this law, and the market value of their stocks and bonds has steadily risen. Also their physical condition is on a

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par with that of railroads in other Southern and Western States, and their incomes from operation are as substantial. The practice of "watering" their securities has been effectually stopped, as regards local issuance, and any interest which might have accrued on such securities has been saved to the public.

It has been contended that the Texas valuations of 1894-95 were too low, and did not, even at that time, represent the fair value of the properties. This is perhaps true to a certain extent, but it must be remembered that the costs of materials and construction then were less than at any time before or since; and, viewed from the present-day standpoint, they seem to have been inadequate. It must also be considered that real estate values throughout the entire State were very low, compared with present values and with those of lands in other States. Although the writer admits that the margin was very narrow, still he is of the opinion that the valuations as made represented closely the cost of reproduction of the physical properties at the time.

The valuations of 1894-95 stand to-day on the Commission's records as "the value of the property," except in cases where there has been application and necessity for re-valuation. The machinery of the law did not provide that these appraisals should be kept "up to date." The mortgages on these railroads are still outstanding, and there has been no call for another appraisal, except in a few instances. The Commission has decided that in its opinion the "present value" of any of the railroads already appraised, is represented by the original valuation plus the value of all permanent improvements and betterments added. This principle has been carried out with those railroads which have applied for re-valuation for any purpose, and the Commission has admitted the same in testimony which it has given before the Courts.

Since the appraisals which the Texas Commission makes are primarily for the purpose of limiting indebtedness, and the carriers are entitled to have these at least equal the cost of their property—the investment with certain additions to cover promoters' profits—no consideration can be given to depreciation of structures and equipment, although the application for valuation and process of issuing of securties may be had several years after completion. The writer holds that there is strong argument in favor of not taking into account "depreciation," and of estimating the value of the property as being entirely "new," whatever purpose the valuation is proposed to serve. This is apparent, as already stated, when the valuation is to serve as a basis for limiting the issue of stock and bonds. Is there any logical reason why a valuation for this purpose should not also serve—as far as it pertains—as a basis for taxation or for regulating freight rates? As far as the State is concerned—and to be consistent—should not "one" valuation serve all purposes?

Suppose that a State should create a board clothed with powers

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of rate regulation, taxation, and authority to restrict indebtedness, and also prescribe that it should appraise the value of the property of the railroads, and use that appraisal as the basis for its acts. Would it be logical for that board to make and apply one system of valuation for one purpose and another system for another purpose? Manifestly, it would have declared that a valuation was a "valuation" for all purposes, at least as far as the physical property was concerned; and, when devising a method for making its appraisals, it should incorporate therein all the elements of value which might apply logically to either purpose. The writer believes that "depreciation" of roadbed and structures would have no place in such an appraisal, on the one hand, nor its negative, but fully as intangible and difficult of concrete estimate, "adaptation and solidification of roadbed," on the other.

It should not be understood that the writer maintains that taxation boards should not go beyond the valuation of physical property to arrive at a final basis for assessment. There are certain intangible elements which should be taken into consideration when taxing property, chief of which is the net income. It is only as far as physical valuations apply in either case that he considers that there should be uniformity.

He does not approve at all of incorporating in an estimate of the physical value of a railroad property such an element as "adaptation and solidification of roadbed," which is credited with so much importance in the Minnesota valuation. In the first place, such an element is incapable of being measured in tangible terms and reduced to a dollars-and-cents basis; second, it cannot be reproduced in the sense that other property is reproduced, and its value does not appear in the capital account of the railroad; and third, it results from the action of the seasons on the one hand, and the working over of the roadbed by the maintenance forces on the other, the cost of which appears in operating expenses. One is constrained to believe that the engineer who insists on incorporating such an element in an appraisal of the physical value of a railroad is hard put to find material with which to swell his estimate. When noting the large difference in value per mile of the railroads of Minnesota, as compared with those of Michigan and Wisconsin—adjoining States—it would appear that undue prominence had been given to this and similar factors.

The writer's experience as appraising engineer for more than 10 years with the Texas Railroad Commission, and for the past 2 years as a construction engineer—having built about 160 miles of railroad in Oklahoma and Texas—confirms his belief that, in the absence of actual figures of cost, right of way and other railroad real estate should be appraised at but little in excess of the market value of abutting property. The practice of the Texas Commission has been to add from 25 to 50 per cent. The conditions under which railroads were built

in Michigan, Wisconsin, Iowa, and Minnesota cannot have been radically different from those in the Southern and Western States. In Texas it has been a rare instance when a railroad has had to purchase all of its right of way. Also, contiguous lands have greatly increased in value since the advent of the railroads. It would appear highly illogical to advocate that these increased values should be multiplied by 3—or even 1½—and used as a basis for taxing the railroads on the one hand, or taxing the public on the other, by permitting indebtedness to be issued against it, the interest on which the latter must pay. The railroad recently constructed by the writer traversed fertile and thickly populated areas, already quite well served with transportation facilities. Only a small fraction of the necessary real estate was purchased by the railroad company, and only in a few cases of such purchase did it pay largely in excess of the market value of the land—and these were where the road interfered with houses and other farm improvements. In cities and towns, land was acquired at practically its fair market value. For rural property, the ratios used by Professor Taylor in the Wisconsin appraisal appear to be quite fair, but in cities they are too high—especially for the Southwest. The Minnesota ratios appear to be unreasonably high. Mr. Thompson.

Any appraisal of the physical value of railroads—in the absence of figures as to their actual cost—is necessarily only approximate, and is correct only within certain limits. Especially with regard to the old roads, where original cost data cannot be had, the values applied to property and construction must be largely speculative. It is impossible to build two railroads in the same territory, on the same specifications, for the same amount; yet, on the basis of cost of reproduction, an appraisal board must apply the same value to each.

The writer believes that unless there is more uniformity as to methods of valuing corporation property, as between the States, all valuations will be more or less discredited, as they should be, by the Courts. It is to be hoped that this paper will be generally discussed by the Profession, and will lead to the adoption of more uniform methods.

Charles H. Ledlie, M. Am. Soc. C. E. (by letter).—The following is suggested as a method of procedure for determining the fair and equitable value of a property: Mr. Ledlie.

1st.—Examine carefully the statutes governing corporations of the class under examination.

2d.—Form an opinion as to whether or not the locality can support such a property, by inquiry regarding the different businesses carried on, bank clearings, railroad facilities, what the surrounding country produces, etc.

3d.—Find from the archives of the company a general description of the property, from its conception to the date of appraisement.

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4th.—By close examination of the minute books, directors and executive committees, there can be ascertained all the details of organization, issuing of stock, bonds and other forms of indebtedness, contracts for equipment, supplies used in the construction, etc.

5th.—Obtain from the general manager or the superintendents an explanation of the details of operating and maintaining the property, including the different classes of service, rates, etc.

6th.—Go over the property, examine it carefully, and talk to any and all employees from whom it is thought that any information can be gained.

The foregoing will give a general knowledge of the property under examination, and will enable one to begin the real work. The examiner's assistants must be competent and experienced men.

7th.—Examine all the vouchers, from the beginning of the company down to the date it began operation; classify their contents under the respective heads for the different classes of material used in the construction, for example, pipe, engines, cable, etc.; then prepare blank tables for each heading, having columns for size, quantity, prices, and total; and abstract each voucher. Do the same with the vouchers for labor, general office salaries, general expenses, interest, taxes, legal, etc. This, when completed, will give the detailed cost, as shown by the vouchers.

Next check the vouchers back through the books, and draw up a statement, which will show the total book cost and, no doubt, will differ from the voucher total. It is likely that many items will be found for which no vouchers exist, a list of these is made and if the officers cannot give a satisfactory explanation of any of them they are omitted. The total of what remains is added to the voucher total and represents the cash expended for the benefit of the original property, as shown by the books and vouchers.

8th.—Take all the remaining vouchers of the company (it is supposed that the examiner has already been informed by the officers, and by his inspection of the records, of the extensions and betterments which have been made), separate the vouchers for materials, labor, etc., from those on operating, etc. Next classify them by years, and then proceed as set forth in 7th, and add the different yearly amounts to the total of the original plant. This will show the amount of cash expended (according to the vouchers and books) on the property, for its physical plant, organization, etc., from its beginning to the date of appraisement.

Every examining engineer should know (or can obtain) the prices for materials, labor, etc., during these periods of original construction, extension, etc. If the prices are the same, or about the same, as at the time of purchase, the above total stands as the cash expended; if there should be much difference—and sometimes there is—take the

detail of the materials as found in the vouchers, affix the proper prices, and do the same with labor, etc., and this total will be what, in the judgment of the examining engineer, the plant should have cost. A mean between this latter total and that in 8th is taken, in order to be fair and equitable. This amount, in place of that given by 8th, is then used as the cash expended on the physical property. If no difference is found in prices, then the total cash as shown by 8th is considered as the total to be hereafter used. Mr. Ledlie.

9th.—A careful detailed inventory is now made of the physical property as it exists at the date of examination. This often requires some excavation in order to determine sizes, quantities, and conditions. The prices used to ascertain the total of the inventory are made by taking the average of all those paid for materials, labor, etc., of the same class, during each year of the property's existence. (The writer considers it manifestly unfair to use the current prices in this calculation, for they may be very much below or above those actually paid, and in either event an injustice would be done, whereas, if the average prices are used, the examiner cannot be accused of unfairness.) To this total cost, as shown by the inventory, 5% is added for engineering and superintendence; 3% for general office salaries, 2% for general expenses, 1½% for legal and organization expenses, and from 5 to 10% for contingencies. (This latter percentage depends on the judgment of the examiner, who, after studying the local conditions carefully, can determine from his own experience what difficulties have been met in the construction. It is not believed that a hard-and-fast rule can, in equity, be laid down for this latter percentage.) This total represents the value of the tangible property, based on the inventory. The inventory cost and that set forth in 8th (or possibly as modified by prices current when the plant was built) are averaged, and this result, plus the supplies on hand, is the fair and equitable amount of cash which has been expended on the property. This is used in finding the "Fair and Equitable Value."

10th.—Next take the inventory of the plant set forth in 9th, affix the current prices at the date of appraisal, and to this total add the same percentages for engineering, etc., as set forth in 9th. This gives the cost of reproducing the property, with the same classes of materials, size and make of engines, etc., as is now in it, to which total add the cost of materials on hand for the total of cash required to be expended at current prices to reproduce the plant as it exists.

It is often found that this latter total is greater than that set forth in 8th, for the reason that the engines, etc., may be of types which are now abandoned or obsolete, and the manufacturing company, having to make patterns, etc., would charge more for them than the original price at the date of purchase.

This reproduction cost at current prices is only to give the examiner

Mr. Ledlie. information he may or may not require later in the investigation to determine some point that might arise in ascertaining the "Fair and Equitable Value."

The writer considers it unfair to call the reproduction value the cost of a modern plant which will give the same service and output, because one is not dealing with the value of a modern plant, but with that of an existing property.

11th.—From this cost (using the detailed inventory to find the extent of property still in existence), calculate the amount of depreciation for each section of the plant, this being based on the present condition of the different parts and what their future life may be. The total depreciation is then deducted from the result found in 9th, and this remainder is used as the "Fair and Equitable Value" of the tangible property at the date of appraisal.

The intangible value (called by many names) must now be determined. It consists of rights, from the State, county, city, or any one or more combined, which the company must have in order to carry on its business. These rights in nearly all States are taxable, and taxes are collected on them. The Supreme Court of the United States has in the past held that they are property, notwithstanding what State "Courts and Commissions" have set forth on this subject, and in the writer's examinations they will be treated as property until the Supreme Court of the United States decides otherwise.

There are in general three classes of franchises, namely:

I.—Those granted by the State to conduct a business, where no county or city franchise is necessary, only requiring the company to obey the ordinances for excavation, etc. The charter of the Laclede Gas Company, of St. Louis, Mo., is an example of this class.

II.—Those granted by the State to carry on a business subject to a county or city franchise.

III.—Those granted by a city to an individual, singular or plural, or a company, to do business within its limits or a section thereof.

In each case the right may be a contract, for it may require a payment for the franchise granted, either in a lump sum or in yearly installments, or in the form of services rendered, such as for light, etc., free service of some kind, or a combination of any two or all of them.

The manner of determining the value of "Intangible Property" is as follows:

11th.—(*a*) The gross collected earnings are audited for each year during the period the company has carried on its business. The same is done for all vouchers, *i. e.*, operating, maintenance, salaries, legal, general expenses, interest, insurance, and taxes, and includes

every item disbursed. Whatever this latter amounts to, is deducted from each year's gross earnings as already found, and the result is the true net earnings or deficit for each year. Mr. Ledlie.

(*b*) The true net earnings are added together and the mean taken; if, in the period from the beginning to the date of appraisement, any deficits are found, these are deducted from the total of the plus-earnings, the result is divided by the total number of years, and this gives the true average net earnings. This is then capitalized at the legal rate of interest of the State in which the property is located. The result is used as the value of the "Intangible Property."

12th.—The amount given by 11th is added to the result obtained by 9th, and this total is the "Tangible and Intangible Value" of the property, and the "Fair and Equitable Value" of the property at date of appraisement.

If it is found that grave mistakes in design or judgment have been made by not employing competent people, and money has been wasted in construction, the plant is re-designed, for the original plant, and its cost estimated. The same is done for each extension, using the prices paid at the different periods, and this result is used in place of 9th, as the cash cost at the date of appraisement.

In determining the intangible value, if it is found that the management has been careless in order to make large net earnings, at the expense of the physical property, estimates are made of what the property can be operated and cared for (here the practical knowledge of operation, etc., is necessary), and these results, plus taxes, etc., are subtracted from each year's collected earnings. The mean or average of these results is considered as the true net earnings, which are capitalized and added as set forth in 12th.

The writer holds that consumers or purchasers should not pay for avoidable error or ignorance, and the amount of the securities issued on the property is not considered as entering into the matter of "Fair and Equitable Value"; when they do, the method is somewhat different.

The mean true net earnings are used in determining the intangible value because franchises have average values, as earnings and expenses fluctuate in corporations, and, when intangible values are to be considered, they must not be based on the last year's net earnings, for if they are, they may give a very large result in one year and a small one in the next; therefore, to be fair, the mean true net earnings should be the basis of the intangible value. If the company has been over-capitalized, and no sinking fund or depreciation has been set aside, it is the present owner's misfortune. If the company calls something a betterment, and it is found that the betterment has only replaced something, it is not allowed, but is classed as maintenance; on the other hand, if the replacement is larger, and capable

Mr. Ledlie. of rendering greater results, such as a larger engine, pipe, cable, etc., the cost, less the cost of what it replaces, is allowed as a betterment, and if the old part is sold the proceeds are deducted from the betterment charge, for if it is credited to maintenance, it increases the true net earnings. This is often done, but is not the correct way to treat the matter, for it increases the intangible value.

13th.—When new rates are to be established for a period of future years, the manner of determining the "Fair and Equitable Value" is the same as has been heretofore set forth. The new rates are based on averages, and the first step necessary is to ascertain what gross revenue the company must have in order to pay all classes of operating expenses, maintenance, depreciation, taxes, interest on the "Fair and Equitable Value" of the property, and a reasonable profit.

To obtain this amount, the procedure is as follows:

(*a*) Find the percentage of increase of the operating expenses for each year over the prior one, for a period of generally five years preceding the date of examination (a longer time may be taken if, in the opinion of the examiner, it is necessary), and then ascertain the average annual increase of the percentages. The result thus obtained is taken as the increase percentage for the operating expenses for the new period of time.

(*b*) In order to determine what the operating expenses will average during the time the new contract is to run, take the amount of the last year's operating expenses as a basis and add to it the percentage found by (*a*). This total is the operating cost for the first year of the new contract. The amount for the second year is found by adding to the cost of the first year the percentage found by (*a*), and so on for each year of the new period. These results are added together and their average is then used as the mean cost of operation for each year during the full period.

(*c*) The same method is followed for maintenance and taxes, in order to find the average maintenance and taxes for the new contract's life.

(*d*) Depreciation on the plant begins from the date of appraisement, and is estimated on the physical property by using for each section the percentages used in determining the "Fair and Equitable Value."

(*e*) Interest at the rate of 6% is allowed on the "Fair and Equitable Value," and 6% profit.

The question of extensions and betterments to the original plant must now be taken into consideration.

(*f*) The amount of the betterments and extensions have already been found for each year of the property's existence, and an average of them is taken as the amount the company will spend on extensions, etc., during each year of the new contract. On this sum 6%

interest and 6% profit is allowed, and, for depreciation, the same percentage as used in the original plant. Mr. Ledlie.

It will be seen that all the expenses of operation, etc., of these extensions have been allowed in (*b*), where the increase has been added for each year for these extensions and betterments, as they are assumed to increase the cost of operation, etc.

(*g*) The amounts found by (*b*), (*c*), (*d*), (*e*), and (*f*) are now added together, and to the sum 5% is added for interest, taxes, operation, etc., which may be caused by the necessary increase in capital expenditures, for a greater growth than could be foreseen at the time the new rates were established, for losses, etc. This total is used as the basis for establishing the new schedule of rates.

14th.—The next step is to determine what part of the amount found by (*g*) must be paid by the different classes of consumers.

(I) First ascertain the yearly percentage of increase in the output of the plant for the five years before the new contract is to go into effect (or longer if, in the opinion of the examiner, it is necessary); then find the average increase of percentage during the beforementioned five years. Add to the last year's output the percentage found above, this result representing the output for the first year of the new contract. Continue this operation for each year in the same way as the operating expenses were found in 13th (*b*). The average of these results will be the average estimated output during the life of the new contract.

(II) Next find the amount of the total output each class of consumers used during each of the five years, and then find the average yearly use during this period. Put these into percentages of the amount of the average output for the five years, and then use the percentages as the amount each class of consumer will use of the average output found in (I) during the period of the new contract.

This gives the average amount of the output each class of consumers will use during the average life of the new contract.

(III) Next find the average percentage of the total revenue each class of customers paid during the five years. Take these percentages as the average percentage each class will pay of the average revenue necessary during the time the new contract is to run.

(IV) Having found the average amount of the required revenue that each class must pay, and the average amount of the total output each class will use, dividing the former by the latter for each case will give the rate each class is to pay during the new period.

It is often found in plants that large extensions have been made to supply a special contract for a long period of time, and these extensions are set aside for the exclusive use of this contract. In such cases exclude the cost, etc., of this part of the plant from the "Fair and Equitable Value" in the matter of adjustment of rates.

Mr. Ledlie. In determining the operating expenses, etc., in such a case, find the percentage of the total output this special output amounts to; then, using this percentage, find what part of the total power-house expenses of all kinds are caused by this special contract. This result is deducted from the total power-house expense, and the remainder is the power-house cost of furnishing the consumers with their share of the total output. If it is found that special employees are required to deliver this special output, their cost is deducted, and the same for the maintenance material used. Taxes and interest on the cost of this special equipment are found by ascertaining the percentage this cost of the special equipment bears to the whole plant.

The above results are deducted from the total operating, maintenance, taxes, and interest disbursements, and "Fair and Equitable Value," and the remainders are used as the cost of the last year's expenses for furnishing the consumers with their share of the product and the "Fair and Equitable Value."

The same method is used in determining the revenue paid by the consumer.

The above result, *i. e.*, cost of operating, etc., is then used as the basis for estimating the expenses for the period of the new contract, as heretofore set forth.

If the charter comes under Class II or III, the city no doubt has incorporated a clause for the adjustment of rates, and the method used above is followed.

15th.—Where the franchise has expired and is going to be renewed, the same method holds.

16th.—Where the franchise has expired and the city has paid a certain amount for service, and is to buy the property, the same method is used, except in determining the intangible value. For determining the latter, the amount the city pays for service is deducted from the gross collected revenue. From expenses is deducted the same percentage as the amount of the city's payment is of the gross revenue; a net revenue is found from this, the taxes paid are deducted, the remainder is capitalized as heretofore set forth, and is the intangible value. Whatever the latter amounts to is added to (or deducted from, in case of deficit) the "Fair and Equitable Physical Value," and the result is the price the city should pay.

Cities generally claim, and so do their "experts," that they should only pay junk value, or the cost of a modern plant to give the same results. This is eminently unfair, because the city buys a property which is in full operation and it receives the full revenue, in addition to obtaining service for itself at a less cost than it heretofore paid. The difference between the cost to the city of furnishing the service itself, and what it paid the company, is profit, but there is a charge against this of loss in taxes. These two latter items, namely,

profit and taxes, generally balance each other, although the writer has known of cases where the city was the gainer. Mr. Ledlie.

There are many points which can be advanced to establish the fairness of the methods outlined herein, but they would take some time to explain, and therefore the writer has only set out the plan he follows in his examinations, hoping that it may be of some aid in establishing a uniform method which will be upheld by the Courts.

It may be stated that recently this method was used in an examination, going back thirty-five years, and the results were accepted by both sides without question.

The writer has refused a number of examinations when told in advance what result must be found, as well as in "Expert" work, where the examiner is expected to help make a case, regardless of his honest judgment, for, by accepting such work, the engineer hurts his reputation and lays the Profession open to such remarks as Judge Lacombe recently made in the case of the Peoria Water-Works Company *vs.* Central Railway Company.

The writer is fast coming to the conclusion that a great deal of legal trouble is caused by the decisions of commissions, the members of which have not had experience in these matters. If a commission consisted of an able lawyer, a financial man, and an engineer who has had a broad operating experience, its decisions would carry weight, and the Courts would not be burdened with so many appeals.

William G. Raymond, M. Am. Soc. C. E. (by letter).—This is, perhaps, the best paper on the valuation of public service property that has yet appeared. The author's analysis is very clear, and his arguments are convincing. Three points the writer would like to consider; two of them briefly. Mr. Raymond.

The item, "going value," even if it is determined on logical reasoning, as suggested by Professor Mead, would seem to be a dangerous item, and one which might result in absurdities when estimated by an unscrupulous, ignorant person. Moreover, the term has been differently defined, and there is no certainty as to just what it means. The writer sees no reason for the existence of such a term, or of such a separate quantity as this term is supposed to represent.

The term, "franchise value," or, "value of the franchise," is used to represent the difference between the capitalized net earnings and the value of the physical property. Of course, there is such a difference, either positive or negative, but there seems to be some objection by the Courts to calling this "franchise value." The writer, therefore, would suggest that, since franchise value is a very elusive item, depending on the life of the franchise, the attitude of the community toward the corporation, the activity of competing corporations, and numerous indeterminate items, the term, "business value," or, "going concern value," be used instead of "franchise value." "Going concern value"

Mr. Raymond. is not as good a term as "business value" or "value of the business," because it may be assumed to include both the value of the business and the value of the property. "Value of the business" would presumably include the value of the franchise, and perhaps would not always be represented exactly by the difference between the capitalized net earnings and the value of the physical property, but would be this difference affected by some judgment percentage resulting from a consideration of the probable continuance of the franchise.

Mr. Riggs has truly said that the value of the physical property must not be made to depend on the purpose for which the valuation is made; that, for the business for which it is used, the value of the physical property is the same, regardless of the purpose for which a valuation is desired; but valuations are made for different purposes, and, while there is room for argument as to the proper valuation to be used for capitalization, taxation, or sale, there are perfectly definite methods suggested for valuing property for these purposes. The writer has never seen a statement—that appealed to him as at all rational—of a proper method of valuing property for rate-making. Indeed, the writer has said* that "proper traffic rates have no relation to valuation except that the minimum net income should be at least sufficient to pay interest on the physical valuation." The writer is not absolutely certain of the correctness of this position, for a study of the public right to regulate a corporation which is performing a semi-public function seems to indicate that the public has a right to say, not only that rates shall be non-discriminatory, but also that they shall be reasonable.

Now, the writer is familiar with three bases for the determination of what constitutes reasonableness of rates. One, which applies to rates as a whole, is this: That the net income should produce not more than a reasonable interest rate on the actually invested capital. Another is the rate that the traffic will bear, and the third is a rate that represents what the service is worth to the purchaser. Of course, a difficulty arises in determining reasonable rates on any one of these three bases.

The only difficulty with the first one is in determining what is a reasonable interest rate on invested capital, and, as far as the writer has read, no Court has yet determined what this is, although some Courts have held that 5% is a not unreasonable return, that 8% is a not unreasonable return, and, if the writer's memory serves him right, that even 15% is a not unreasonable return.

There is great difficulty in the determination of what the traffic will bear. It is a matter of the exercise of judgment and of experiment, and must be applied to a considerable extent to particular rates, for particular commodities, for particular places.

* "Elements of Railroad Engineering."

The third basis would seem to be the most difficult to use, although it is one which has recently been established in important Court decisions, and is mentioned by Mr. Riggs. What is a monopoly-provided service worth to the user or purchaser? Suppose that a gas company charges $1.60 per 1 000 cu. ft. for gas, and a very considerable part of the populace living in the city served purchases gas at this price. Presumably the purchasers pay what the service is worth to them, and what they are willing to pay rather than suffer the inconvenience of tallow candles, oil lamps, or to pay a high price for electric lights. Suppose that through a period of five years, by a series of reductions voluntarily made, the price of gas finally reaches $1.15 per 1 000 cu. ft. Is this gas worth any less to the consumer at the end of the five-year period than it was at the beginning? So far as the writer can see, it is, for only one reason, namely, that it can be had for less; but this has been a voluntary reduction on the part of the supply corporation, and who shall say that the service is not worth less than $1.15 to the consumer, or who shall say that it was not worth less than $1.60 at the beginning of the period suggested? The figures here given represent an actual case which has occurred during the last five years, within the writer's knowledge. There seems to be a growing feeling among the people that rates as a whole must be fixed so as to yield only a reasonable return to the corporation, and, apparently only for want of the suggestion of a better method, a reasonable return has been held to mean a reasonable return on the capital invested. Believing that there may be some ground for the claim that rates as a whole should be thus fixed, and that the return should not be unreasonable, let us consider how what is reasonable may be determined. Mr. Raymond.

In the first place, it appeals to the writer that the invested capital is not the proper basis for estimating reasonable rates. If it shall be finally established that a corporation is entitled to realize only a reasonable interest rate on the capital invested, there will be no more public service corporations organized; but, if the reasonableness of the return may be based on the capital invested and the business done, there will still be good inducement to capable men to engage in public service business.

It would seem that the rate of return that is reasonable differs for the capital invested and for the business done—that is to say, if the capital invested is $1 000 000 an ordinary investment return of from 4 to 5% may be sufficient; and if the business done with this million-dollar plant amounts to $10 000 000 a year, a reasonable return may be 10% or even 15% of the whole.

Now, as has been suggested by Mr. Riggs, it is manifestly impossible to capitalize the net earnings as a basis for determining reasonable rates, because these net earnings are the result of certain

Mr. Raymond.

rates already established, the reasonableness of which may be in question; and if, instead of speaking separately of interest rate on capital actually invested and profit rate on business done, it is desired to obtain a value on which to base reasonable rates, the following is suggested as a method: Determine the physical value and the annual interest on this physical value at an assumed reasonable rate, say 5%; determine the annual expense of conducting the business, and assume a business man's profit rate, say 15%, and find the profit that should be earned on the business done. This, added to the total interest charge, should give the net income, over and above operating expenses, that may be considered reasonable, and this sum, capitalized at any given assumed reasonable interest rate, would give a value which might with reason be used as a basis for rate-making, rates being deemed to be reasonable as a whole which furnish from year to year a simple reasonable interest rate on this established value. Of course, there is no necessity for establishing such a value, as the reasonableness of the rates will be determined when it is learned that they produce not more than a fair interest rate on the actual physical value of the property plus a fair profit rate on the business done.

This method is not free from the objection that what is a reasonable interest rate and what is a reasonable profit rate have never yet been fixed, but it is much easier to fix these separately than to fix what is a reasonable return on the capital actually invested or the physical valuation of the property.

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PAPERS AND DISCUSSIONS

TIMBER PRESERVATION, ITS DEVELOPMENT AND PRESENT SCOPE.

Discussion.*

By Messrs. Richard Lamb and J. Martin Schreiber.

Richard Lamb, M. Am. Soc. C. E.—There has been no improvement in materials or processes for timber preservation since the Special Committee on Wood Preservation, of this Society, reported in 1885. Inventors and merchants have patented so-called improvements on the Burnettizing or zinc-chloride process, the Kyanizing or bichloride of mercury process, the Thilmany or sulphate of copper process, and the Bethell or creosote in hermetically-sealed cylinder process. To an engineer familiar with the business of wood preservation, these so-called new processes seem to have been designd mainly for promotion purposes, to secure a trade advantage rather than additional efficiency, or additional economy without loss of efficiency. Mr. Lamb.

With the exception of the business of treating wood blocks for paving, none of the new materials in combination with the tried ones, has made any considerable headway. It will be conceded that the Bethell process, or creosote in hermetically-sealed cylinders, has exceeded all others in general use for preserving wood against all causes of deterioration. It stands alone for use in preserving piles against the ravages of the *Teredo Navalis*. Opinion has not changed as

* This discussion (of the paper by Walter Buehler, M. Am. Soc. C. E., printed in *Proceedings* for November, 1910, and presented at the meeting of December 21st. 1910), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lamb. regards the need of naphthalene in the creosote oil for this purpose, that attribute being the main stable germicidal agent of creosote oil.

The Burnettizing or zinc-chloride process is still used to a considerable extent for railroad ties where the roadbed can be kept comparatively dry. This process is cheaper than creosote, and quite effective, but the zinc chloride washes out of the wood in wet locations. Engineers who are advocating preservatives which contain no antiseptics or poisons should note that zinc chloride, bichloride of mercury, and sulphate of copper depend solely on poison to preserve the wood. Tar acids and naphthalene are the only poisons in dead oil of coal-tar. To leave out these fractions from creosote would be to abandon the only known method of preserving wood, namely, poisoning the bacteria of decay. In a paper on the preservation of ties, before the New York Railroad Club,* the speaker showed that creosoted ties pay for themselves every 15 years.

Since the presentation of the report of the Special Committee of this Society, creosoted conduits which had been in use 21 years have been taken up and proved to possess 100% salvage value; a number of these conduits have been relaid.

There no longer remains a question as to the feasibility or advisability of preserving wood. There is much talk in the United States now about the need of the conservation of the forest supply. This can best be accomplished by treating or preserving the wood after it is cut, so that a less quantity will be needed to meet our requirements. Woods which without treatment would decay too quickly to be of commerical value become valuable when preserved, and engineers should study well this phase of the subject.

The conservation which should be sought now is that of the waste from the vast number of coke ovens in the United States. We import 38 640 000 gal. of creosote oil per year for wood preserving, only getting 17 360 000 gal. from home sources. In Europe they apply condensing apparatus to coke ovens, by which waste products heretofore lost are liquefied, producing a creosote like the distillate of coal-tar procured in America from gas-houses using coal. Much of this creosote is shipped to the United States.

In order to determine the best attributes of coal-tar for preserving, the speaker knows of no surer method than to take samples of treated wood of known history and make analyses by fractional distillation of the oil remaining in the wood. This has been done by the United States Forestry Department, and the results are recorded in Circular No. 98.

The averages in Table 1 have been compiled from the Government analyses of all tests.

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* *Proceedings*, New York Railroad Club, February 18th, 1910.

TABLE 1.

Mr. Lamb

Samples.	No. of samples.	No. of years in service.	Creosote, in pounds per cubic foot.	Percentage, Distillation of Extracted Oil.				
				205° to 245°	245° to 270°	270° to 320°	320° to 420°	Residue above 420°
Cross-ties	20	21.08	9.50	9.65	13.77	24.	24.69	22.46
Piles	12	32.	12.39	20.59	15.53	19.77	18.82	22.48
Paving blocks	5	20.6	13.77	20.92	22.96	23.32	16.56	15.58

The following are the usually accepted fractions for the degrees of heat, centigrade, as recorded:

Light naphtha, up to		110°
Light oil,	commonly called tar acids	110° to 170°
Carbolic oils	commonly called tar acids	170° to 225°
Creosote oils		225° to 270°
Anthracene oils		270° to 360°
Tar or pitch		above 360°

The Government officials, however, have selected an arbitrary division for their fractions, as recorded in Table 1. However, by applying the foregoing commonly-used divisions, an idea of the nature of the various fractions can be secured.

The tests showed that piles which endured had been preserved by an oil which left in the wood an average of nearly 26% of naphthalene, and one sample showed 48 per cent. They all also had a good percentage of anthracene, which is undoubtedly of equal importance with naphthalene as a preservative.

The results of tests also showed that from 10 to 12 lb. of creosote per cubic foot is ample for railroad ties, and that from 10 to 20 lb. is needed for piles, depending on whether they are used in the warm waters of the South, where the *teredo* is more destructive, or in the colder waters of the North, where the season of their boring is shorter. Naphthalene and anthracene fractions predominated.

The average quantity of creosote taken from paving blocks which had been in use 20, 29, and 34 years, was 16.14 lb. The naphthalene and anthracene percentages were conspicuously large.

Tar acids or volatile oils are strong antiseptics. Although their stay in the wood is comparatively short, they exert an antiseptic influence on the bacteria of decomposition, and render a valuable service thereby. The speaker is convinced, however, that we have all the antiseptics needed in the naphthalene and anthracene which remain in the wood, and that not more than 1.5% of tar acids should be admitted in the creosote oil used for wood preservation.

Mr. Lamb. While it is important not to accept any oils distilling below 150° cent., in order to avoid the carbolic or tar oils, which evaporate from the wood, it is equally important that no product of distillation should be accepted above 360° cent. This product represents simply pitch, which has no antiseptic qualities or ability to preserve the wood, other than to caulk the pores. This pitch is practically the same as that in heart pine, and will not preserve the wood longer than the natural pitch, so far as it acts as a germicidal agent. It requires high heat to keep it sufficiently liquid to enable it to be injected into the pores of the wood. This heat breaks down the fiber tissues, and takes the life out of the wood. Persons familiar with kiln-drying timber know that if it is dried at 280° Fahr., it becomes too brittle to dress. Where blocks have been treated with pitch, or oil of high specific gravity, say, 1.10 or 1.12, and laid in places where the foundations were not solid or the blocks not rigidly held, as in the case of a certain bridge in Brooklyn, and the floors on the ferry-boats of the Central Railroad of New Jersey, the blocks will be found to be split almost into splinters. In a bridge in Baltimore, in which the conditions were similar to those in the bridge just referred to, blocks of short-leaf pine treated with regular creosote oil were laid 6 years ago, and are as good to-day as when laid, in spite of the very heavy traffic which it has carried. Regular creosote oil, having a specific gravity of from 1.03 to 1.07, was used in treating those blocks, therefore no excessive heat was necessary to inject it, and the wood remained intact as to its fiber adhesion, no blocks having split.

It is contended by many engineers that this pitch or oil of 1.10 specific gravity is preferable in the case of paving blocks, because it prevents the absorption of water. The sole purposes of stopping the absorption of water are: to prevent the alternate wetting and drying (which is a primary cause of decay) and the expansion of the blocks (which causes the buckling of the pavement). As a matter of fact, this heavy oil does not penetrate the blocks as thoroughly as the lighter creosote oil, and when they are split for closure-blocks it causes the exposure of the interior where the moisture can enter the wood. However, if creosote oil will preserve the wood, it is idle to contend that the pitch is better as a preservative, because it will caulk the wood better than creosote oil. If a paving block treated with 20 lb. of pitch per cubic foot is dried, using 100° heat for 24 hours, and is then immersed in water, it will not absorb more than $3\frac{1}{2}\%$ by weight. Creosote oil having a specific gravity ranging from 1.03 to 1.07 will absorb about 4 or $4\frac{1}{2}$ per cent. The latter, however, penetrates the wood more thoroughly, coating the fiber, which is the part that absorbs the water when the swelling of the wood takes place.

The Lowny process is based on the principle that it is not neces-

sary for the oil to remain in the wood after it has coated all the pores thoroughly, and means are provided for extracting the oil after it has been injected. Although the speaker does not believe that this patented process will have very extended use, because it is objectionable to dilute the oil with the sap and steam and then use it over again; yet the principle is based on reasonable grounds, for experience has proven that after a few years' use one never finds in the wood the same amount of oil as was injected into it, and still its life is extended without appreciable deterioration. Mr. Lamb.

In regard to the buckling of wood blocks, the speaker's observation has shown that when they are treated with the real creosote oil, this does not occur. He has never seen nor heard of the buckling of any properly creosoted wood blocks. On the other hand, in a large number of cases, he has seen serious buckling with blocks treated with pitch of 1.12 specific gravity.

The kinds of creosote are dependent on the sources of supply, which may be named as follows:

(1) Wood creosote oil,
(2) By-product coke-oven distillate,
(3) Water-gas tar distillate,
(4) Coke-oven distillate, and
(5) Coal-gas tar distillate.

These are numbered in the order of their value as sources for getting the oil containing the best constituents for wood preserving, as determined from the results of tests of woods treated, compared with the length of time they would have lasted without treating.

The speaker was associated with the Carolina Oil and Creosote Company, probably the largest plant ever built for distilling wood for wood creosote oil and preserving timber therewith. Time showed that this oil could not be depended on for protection, either against the *teredo* or decay, and the process was abandoned.

Of late years by-product ovens have been perfected, and the Otto-Hoffman process has been established in America to make various products from the distillation of coal, but because of the high cost of the plants, its general introduction has been slow.

These by-product plants make coke, illuminating gas, sulphate of ammonia, and by-products of coal-tar. The residuum, after the foregoing valuable products have been extracted, is pitch of 1.12 specific gravity, containing but little naphthalene and anthracene. This pitch is used for roofing and water-proofing, and one company, with its allied interests, as far as can be ascertained, has bought up all the output that can be used for wood-block paving.

A committee of the American Society of Municipal Improvements recently reported that an oil of at least 1.10 specific gravity, at a

Mr. Lamb. temperature of 38° cent., should be used for preserving paving blocks; and stated that its investigations had shown that this oil is not controlled by one company.

One of the large creosoting companies manufacturing paving blocks wrote to each member of that Committee, stating that for some time this country, as well as foreign countries, had been canvassed for a delivery of 100 000 gal. of this oil, and it was found that it could not be obtained. Each member was asked to forward the names of companies in this country, in Canada, or other foreign countries, which manufacture this oil, and have the capacity to furnish it in large quantities—from 50 000 to 200 000 gal.—in a reasonable length of time after the order is placed. The answer to this letter, signed by the Committee, states in part:

"We might say, to start with, that the assertion in the report of the committee of the American Society of Municipal Improvements on the subject of Wood Block Paving Specifications that 'investigation on our part had shown that the oil recommended was not controlled by one company' was made in reference to wood block manufacturers only, and did not relate to oil manufacturers."

The only company mentioned which could furnish the oil is the one which controls the output of the 1.10 specific gravity oil. This company furnishes the 1.03 specific gravity oil to the creosoting company referred to, but has always refused to furnish the 1.10 specific gravity oil.

The chairman of the Committee wrote that a letter just received from Dr. Gellert Alleman, Professor of Chemistry, Swarthmore College, states:

"I have no hesitation in stating that any dealer in tar can fulfill the specifications to which you refer by either filtering the tar and adding a certain proportion of creosote oil to it, or in some cases by adding creosote oil to the unfiltered tar."

Attention is drawn to the speaker's remarks* on this phase of the subject in his discussion on Chicago paving practice.

One member of the committee wrote:

"I have delayed answering your letter for the reason that the question of the limiting value of the specific gravity of creosote oil was referred to the Chairman of this Committee, and I, personally, did not look into this subject. Furthermore, the time allowed this Committee for the preparation of a report did not permit of an investigation into this subject by each of the members of this Committee."

The speaker has no doubt of the good faith of this Committee, and in view of the fact that the combination of wood-block manufacturers who can command that special oil has done practically all the wood-block paving propagation during the past few years, the members of

* *Transactions*, Am. Soc. C. E., Vol. LXVI, pp. 43-48.

the Committee, as well as many of the best engineers, have been misled in this important engineering business. Mr. Lamb.

The specifications of this Committee require that the distillate shall not exceed 2% up to 150° cent., and shall not exceed 35% up to 315° cent. This is strictly a call for the by-product pitch, and is a bid for an oil which can be absolutely void of naphthalene and anthracene, the materials which engineers have depended on thus far to preserve the wood. It is begging the question to say that the kind of oil is immaterial, because it is expected that the pavements will be replaced before they would naturally have time to rot. If this is so, why specify other than an oil that experience has shown will preserve wood, unless it is a question of cost? As a matter of fact, although creosote costs more than pitch, when the Bridge Department of the City of New York specified for the Manhattan Bridge the regular blocks treated with creosote oil, there was a very material drop in the cost of the pavement under that of former orders when the 1.12 specific gravity oil was specified. Richmond is the only Borough of New York City in which regular creosote oil is specified. Learn what drop in prices there was when the specifications were changed from 1.12 specific gravity oil to regular creosote oil blocks, and compare the higher prices for similar work done in the other Boroughs, which call for the heavier oil.

Water-gas tar or pitch is as good as the by-product oil; in fact, it is very similar. It is more plentiful; any one can buy it. In a valuable paper by Charles N. Forrest, Assoc. M. Am. Soc. C. E., on "Preservatives for Wood Paving Blocks,"* by-product tar and water-gas tar are compared. He states:

"From the data already presented there appears to be no reason why coal-tar should be better than water-gas tar for the preservative treatment of wood paving blocks, but the following comparison of creosote oil distilled from water-gas tar with creosote oil distilled from coal-tar will undoubtedly be of interest to engineers who are interested in the preservative, aside from the water-proofing, features of this tar."

The following is the comparison of coal-tar creosote and refined water-gas tar referred to:

	Coal-tar creosote.	Refined water-gas tar.
(1) Original specific gravity at 15° cent.....	1.04	1.14
(2) Fraction distilling below 315° cent......	84.8%	16.00%
(3) Fraction distilling above 315° cent.......	13.8%	54.3%
(4) Total distilling from oil................	98.6%	70.3%
(5) Coke remaining in retort..............	1.4%	29.7%

* *The Engineering Record*, April 16th, 1910.

Mr. Lamb.

The comparison of the three oils is as follows:

	Creosote oil.	Coal-tar called creosote.	Water-gas tar.
(1)	1.04	1.122	1.14
(2)	84.8%	34.2%	16.0%
(3)	13.8%	41.8%	54.3%
(4)	98.6%	76.0%	70.3%
(5)	1.4%	24.0%	29.3%

In buying creosote oil for wood preservation, we seek to purchase oils containing as little coke as possible. Note the above percentages of coke. The fractions distilling below 315° are naphthalene and anthracene.

In view of the past failures of water-gas tar as a wood preservative, the question is: What engineer wishes to specify that oil, or what company wishes to guarantee it? If it is a hazard to use water-gas tar, why is it not also a hazard to use by-product tar, which is practically the same? If specifications did not demand that the oil shall be the product of coal, either oil could be used under the specifications.

This Society should appoint a Special Committee to investigate this subject thoroughly and independently, and compile specifications strictly from an engineering standpoint.

The American Railway Engineering and Maintenance-of-Way Association adopted specifications for creosoting, demanding the best obtainable grade of coal-tar creosote—the pure product of coal-tar distillation, free from admixture of oils or other tars. It requires that the creosote shall have a specific gravity of at least 1.03 at 38° cent.

The speaker believes that the most desirable specification for creosote for all wood preservative purposes is as follows:

The creosote shall be not less than 1.03 specific gravity at 38° Fahr., and not more than 1.07 specific gravity, the latter in order to insure thorough penetration. A fractional distillation, using 100 grammes of creosote, shall show percentages of dry oil by weight about as follows:

Up to 150° cent. (302° Fahr.), no distillate.

Between 150° cent. (302° Fahr.) and 170° cent. (338° Fahr.), not to exceed 1.5%.

Between 170° cent. (338° Fahr.) and 235° cent. (455° Fahr.), not to exceed 35%.

Between 235° cent. (455° Fahr.) and 300° cent. (572° Fahr.), not to exceed 35%.

The residue should be soft. The oil should contain not less than 25% naphthalene and at least 15% anthracene oils. Of the creosote oil, 95% should be soluble in carbon bisulphide, and equally in absolute alcohol. This test is to determine the amount of coke.

The above specification would have met the requirements for a creosote as good as any used in the tests as reported in the Government circular previously referred to. Mr. Lamb.

If a specification for creosote was standardized by this Society, the coal-gas manufacturers would make their creosote conform to it with sufficient closeness, and the many bee-hive coke oven operators could be induced to distill their waste product for this specific oil. As it is now, many specifications are demanded, and there is no assurance that any one formula of oil a company might produce would command a market.

It is a fact that neither the by-product oil nor the gas-tar oil has been tried for wood block paving for a longer time than good heart pine will last. Heart pine blocks have always been specified when by-product oil has been called for. If real creosote is used, short-leaf pine can be specified, at a saving in cost for the wood of at least 15 cents per sq. yd. for blocks 3½ in. deep.

Nelson P. Lewis, M. Am. Soc. C. E., in a valuable pamphlet, has given an account of the European practice in wood block pavement, in which he shows that European nations have found the softer woods more durable than the harder ones. Baltic pine, which is like our short-leaf pine, is generally used abroad.

Data concerning wood block pavements have been collected recently by the American Society of Municipal Improvements, from the engineers of a number of American cities. Under the caption, "What Wood is Preferred," Boston, Mass., reported: "Short-leaf pine. Also long-leaf." Boston was the first city in the Eastern States to take up modern wood block paving, all the first blocks being made from long-leaf pine, as nearly all heart as could be secured. It is interesting to note that the engineers of that city are among the first in America to decide that short-leaf pine would be preferable.

The speaker has endeavored to show that pitch oils as wood preservatives can only be considered as experimental. In spite of this fact, the engineers of America are laying hundreds of thousands of dollars' worth of wood block pavements yearly, specifying blocks treated with that material, when they have the experience of Europe, California, and Texas to teach them that in those places regular creosote oil has satisfactorily preserved paving blocks for more than a quarter of a century.

J. Martin Schreiber, M. Am. Soc. C. E. (by letter).—Mr. Buehler's paper is a conservative résumé of the more important requirements of modern wood preservation. The great attention that is being given to this subject is certain to produce a more correct understanding in the future. The whole proposition of wood preservation, with its numerous ramifications, requires a peculiar combination of special study. We should have, not only a thorough knowledge of the Mr. Schreiber.

Mr. Schreiber. characteristics and performance of the wood, but also the chemistry of the preservative and the mechanical knowledge of plant operation. Lastly, the commercial instinct, or experience for purchasing the supplies, is necessary, in order to put the whole proposition on an economical and practical basis. Probably there is no better indication of the necessity of differentiating the subject into component parts and studying carefully each division than the large variation in the results of treatments that have been carried out, supposedly by the same standard process.

About a year ago, the writer's attention was called to the failure of a structure in which the timber had been impregnated with 20 lb. of oil per cu. ft. The trouble appeared to be that the original oil was of inferior quality, so that a large proportion of it disappeared after three years. The practice is rapidly passing of blindly plugging a stick full to saturation, irrespective of classification, the seasoning of the wood, and the exact quality of the preservative, leaving the principle of waterproofing finally to accomplish all. However, some modern specifications are still very unique in their generalities, for instance, the following is quoted from a specification for the treatment of wood paving blocks for a city of considerable size:

"The blocks will be treated by the injection of an impermeable and antiseptic mixture, 22 lb. per cubic foot, that contains at least 50% of inert oil of soft bituminous coal-tar or dead oil of coal-tar. The rest of the mixture will be composed of material adequate to offer impermeability to water."

What the contractor could be compelled to use for "the rest of the mixture" it would not be an easy matter to ascertain.

In relation to methods, it may be of interest to state that Mr. E. H. Hartman has applied for a patent for a combination process in which the seasoned wood is first placed in a hot bath of creosote oil and then treated in a cold solution of zinc chloride. The principle may be used with either the pressure or open-tank methods. It is unlike the Card process, as it is a two-movement method, and is different from the Allerdyce, inasmuch as the oil treatment is given first, instead of the solution of zinc chloride.

Probably the author has not given the light treatments with high-class oils the prominence that they deserve. Considerable development has been made in the empty-cell process, for both the pressure and the open-tank methods. Some prominent railroads are treating ties by the empty-cell pressure method, in spite of the fact that advocates of other processes have assailed the principles involved. The empty-cell pressure process seems to be of considerable importance, not only on account of the deep penetration obtained at a moderate cost, but because the wood comes out of the treating cylinders comparatively dry. Oil dripping is objectionable, not only on account of waste, but also

because it often gives trouble after the lumber has been installed. In the 1910 Report of the Committee on Preservation of Poles and Cross-Arms, of the National Electric Light Association, an empty-cell process for the open tank is described. Briefly, the timber is treated by the ordinary hot and cold process in creosote oil, and then a third step is taken by reheating the timber in oil; this results in driving a portion of free oil from the wood. Here again is obtained the benefit of deep penetration with a limited quantity of oil. The importance of a proper quality of creosote oil in any empty-cell process is readily apparent. Mr. Schreiber.

One of the most pertinent subjects at present, in connection with wood preservation, is that of water-gas tar creosote *versus* coal-tar creosote. In 1908, approximately 17 000 000 gal. of coal-tar creosote oil were produced in the United States, while 39 000 000 gal. of the coal-tar creosote used was a foreign product. The scarcity of the domestic oil will undoubtedly continue, at least as long as the present conditions of by-product industry obtain. The quantity of water gas made in the United States is about twice as great as that of coal gas, and the creosote made therefrom is similar in many respects to that from coal-tar.

The writer was recently surprised to obtain from a contractor a bid for treating cross-ties in which the prices quoted were higher for water-gas creosote than for coal-tar creosote. It was understood, of course, that each oil was to have the same physical properties. This does not seem to indicate the alleged advantage, at least at present, and from the commercial standpoint, of using the water-gas creosote as an adulterant or substitute.

Some interesting papers have been presented recently, before the American Branch for Chemical Industry, by S. R. Church and J. M. Weiss, and with the discussion have considerable bearing on the question of the value of water-gas creosote as a preserving agent. Mr. Weiss, whose paper had the caption of some experiments on the action of oils and tars in preventing mould growth, said in conclusion that the coal-tar creosote, as well as the undistilled tar, is antiseptically more efficient in about the proportion of 6 to 1. It is unfortunate that the chemistry of creosote oils is not well enough known to enable one to form more definite and authentic opinions on the value of the creosote oils, other than their physical characteristics, and it is hoped that further data will be obtained along this line. Although the antiseptic ratio between the oils of coal-tar and water-gas tar may not be as important where the timber is treated by the full-cell method, it certainly requires serious consideration where the treatment calls for a limited quantity of oil.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES EDWARD GOAD, M. Am. Soc. C. E.*

DIED JUNE 10TH, 1910.

Charles Edward Goad was born in London on March 15th, 1848. He attended the College of Preceptors, in London, passing his final examinations with honors in mathematics. Later he received the degree of Associate of Arts at Oxford University. Until 1869 he was engaged in the building of various public works in England, and in that year he went to Canada. From 1869 to 1873 he was Assistant and then Division Engineer on the Toronto, Grey and Bruce Railway. From 1873 to 1875 he was Locating Engineer and in charge of the drafting office for the land and structure plans, and then Engineer for the contracting company building the Montreal Northern Colonization Railway, afterward the Quebec, Montreal, Ottawa and Occidental Railway.

In 1876 he commenced the compilation of a series of special surveys of Canadian cities and towns for the use of Insurance Companies. In 1877 he became Chief Engineer of the Halifax and Cape Breton Railway, but in the following year found it essential to the success of the system of insurance plans which he had originated to give his whole time to the supervision of that work.

Some idea of the extent to which his system has developed may be gathered from the fact that the surveys now include all the important areas in London, England, from Kingston-on-Thames to Gravesend, fifty-five of the principal cities and towns in the British Isles, several cities on the Continent of Europe, in the "Near East," South Africa, the West Indies, and South America. In Canada more than 1 300 places, from Halifax to Vancouver, have been surveyed and mapped.

In 1909, while engaged on a survey at Valparaiso, Chile, on behalf of the Fire Offices Committee, Mr. Goad had a paralytic stroke. As soon as he could be moved he was taken to England, the voyage being of much benefit to him. After a short stay, he returned to his home in Toronto, with his health so far improved as to encourage the hope that he would be spared for a number of years, but after a journey South and a short stay in Florida, he returned to Toronto in April, 1910, where he died from the effects of a second stroke of paralysis. He is survived by his widow and eight children, three sons and five daughters.

* Memoir prepared by the Secretary, from information on file at the House of the Society.

Fire insurance men will remember Mr. Goad as the author of "Fire Plans for Cities," and as the founder of the business now carried on as a Company under the title of Charles E. Goad, Limited. In 1881, in Montreal, he founded the publication, *Insurance Society,* but, a few years later, on account of the increasing demands on his time, he transferred it, the paper developing into the *Chronicle,* devoted to banking, insurance, and finance.

Mr. Goad was a man of sterling integrity and indomitable energy, and never happier than when he had difficulties to surmount. Whenever he had opportunity to give any of his time to the public service, notably as a Member of the Executive Committee of the British Fire Prevention Committee, and as an Advisor on the Guild of Civic Art in Toronto, he was unsparing in his efforts for the furtherance of the objects they had in view.

Owing to the wide area of his professional and business interests, he was engaged in almost constant travel, and so did not have the opportunity of building up that large amount of local influence and popularity usual to men of his type who retain one center, but his social acquaintance was very wide, and he was equally popular on both sides of the Atlantic.

To those who were intimate with him, he was endeared by the many fine qualities he possessed. He was entirely lacking in ostentation, and warm-hearted and loyal in his friendship.

His eldest son, Mr. Charles Ernest Goad, who for some years has had charge of the British and Continental surveys, succeeds him as Managing Director of Charles E. Goad, Limited, of London, England, and will also undertake the direction of the Canadian business.

Mr. Goad was elected a Life Member of the Royal Canadian Yacht Club about ten years ago, and was a very ardent supporter of its interests, though he had not the leisure to participate very much in the sport.

Mr. Goad was an active Member of the British Fire Prevention Committee, a Fellow of the Statistical Society of England, and a Member of the London Chamber of Commerce. He was also a Member of the Engineers' Club of Montreal, the Engineers' Club of Toronto, the Engineers' Club of New York, the Toronto Club, and the Canadian Society of Civil Engineers. He was elected a Member of the American Society of Civil Engineers on September 7th, 1881.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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PAPERS AND DISCUSSIONS

THE GOING VALUE OF WATER-WORKS.

By Leonard Metcalf and John W. Alvord, Members, Am. Soc. C. E.

To be Presented April 5th, 1911.

In any engineering project involving ultimate return to the investor in the shape of revenues, there must be considered not only the expenditures necessary to build the physical plant—with which, of course, the engineer is most directly concerned—but also the investment necessary to put the plant into successful operation and to create the revenues that justify its construction.

This portion of the cost of a project has been termed by the Courts, the "Going Value," or the "Going Concern Value."

Too often the promoters or investors in a legitimate enterprise have either overlooked the importance of providing the necessary funds to cover this cost of the Going Concern, which it will be seen is made up largely of the deficits occurring in the early operation and development of the business of completed plants, or they have under-estimated their amount. In the same way, also, the engineer has failed sometimes to call the attention of the financier to the fact that such deficits will inevitably be encountered in the early days of operation. Such costs not being properly chargeable to physical construction, the engineer, called in by his client to design and supervise the building of the plant, but not concerned with its operation or with the upbuild-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

ing of its business thereafter, has naturally considered these costs to be outside of his domain or responsibility.

There are circumstances, however, which compel the engineer to give attention to this part of the investment, and which raise the question whether, after all, he is not better fitted to determine these expenses intelligently than is the promoter, the financier, or the owner; for the study of such added values involves primarily knowledge of plant operation, as well as business promotion, a class of information which the consulting and supervising engineer must have, in order to design plants which may be operated economically. The engineer, well versed in such matters, therefore, should be as fair and competent a judge of the going value of a project as of the design and cost of its physical structure.

The necesssity of estimating going value has been forced directly on the attention of the engineer engaged in constructing and operating public utilities by the fact that he is often called on to value the plant and property of a "going" concern. The work of estimating the physical part of the property is not difficult, and usually lies well within the domain of his acquired experience, but when it comes to estimating the value of an already acquired income, involving, as it does, a working knowledge of operating costs, hypothetical growth, estimates of semi-developed revenues, and the like, the engineer finds himself confronted with a problem which demands—in addition to such experience—some knowledge of the law, a judicial attitude of mind, and logical thinking of the highest order.

In the last dozen years or so, the frequent demands for the appraisal and valuation of water-works properties in America, for the purpose of purchase or for rate-making, have compelled the attention of engineers to this difficult subject to a degree perhaps greater in water-works than in any other class of public utilities, so that, while going value is common to almost all forms of human enterprise, and will inevitably be more and more clearly recognized in all cases where complete preliminary estimates or scientific re-valuation are required, yet, up to the present time, water-works valuations have been responsible for most of the effort which has been made to systematize logical investigation of this subject.

Methods for properly estimating going value have been discussed in water-works valuations for more than a dozen years, and many

minds have contributed valuable suggestions which have modified early views and systematized its computation, until now certain general principles, concepts, and courses of procedure for determining going value, have come to be recognized as sound and helpful.

It is not strange, however, that estimators new to the subject, who from time to time find themselves obliged to give it attention, are temporarily involved in some of its complexities, and originate suggestions which have already been well discussed and perhaps shown not to stand the test of reason. This, of course, tends to confuse the discussion. It is desirable, therefore, that what has already been accomplished should be recorded, so that further progress can be made.

In an attempt to record what had already been thought out, as well as to suggest lines for further discussion, there was presented to the American Water-Works Association, in June, 1909, a paper by Mr. Alvord entitled "Notes on Going Value and Methods for Its Computation," to which those interested may refer for a more general and preliminary treatment of the subject. It may be added that a valuable discussion followed the presentation of the paper.

Fully recognizing that lack of specific information and discussions have given rise to honest differences of opinion as to the details of going value determination, the writers offer this paper as a further contribution to the subject, with particular reference to a certain phase of it, namely, the consideration to be attached to the interest account. This item, though of comparative insignificance in the valuation of small plants, is of very substantial importance in its bearing on the going value of large plants.

Definition of Going Value.

Going value may be defined as the value of a created income, or —from the reproduction point of view—the cost of acquiring a given income. Commonly, though erroneously, considered as an intangible value, it is none the less a real element of cost, which must be taken into acccount.

The "going value" of the public service or quasi-municipal corporation is somewhat analogous to the "good-will" of the business corporation. It requires time and investment to build up good-will in business. The operator of the public service corporation knows that, under ordinary circumstances, he cannot hope to meet the operating

expenses and fixed charges of his corporation in the first few years of its existence. It must pass through a formative period during which the losses in operation and fixed charges must be carried until its gross income is sufficient, not only to meet the operating and maintenance expenses, but also to leave a margin sufficient to cover depreciation and fixed charges.

Going concern cost, therefore, is as much a part of the reproduction cost of a public service corporation's property as is the reproduction cost of its physical plant. It appertains, however, to the business, rather than to the physical part of the plant.

As a fundamental principle, it must be clearly understood that cost is not synonymous with value. What a property has cost in the past may be of use in determining what its present value is, but clearly past cost is not necessarily identical with present value; therefore, in determining past cost one should confine oneself to that line of inquiry alone until it is completed, and in reproducing a plant it is only proper and logical to consider similarly what it would cost to rebuild it under present conditions. The two should not be confused. Both may aid in determining value. Neither is value of itself alone.

The Wisconsin Public Service Commission, in one of its recent decisions, has taken as a measure of going value, in a reproduction of the property, the past cost to the company, in operating deficits, advertising, and other expenses incident to building its income up to the point where it is sufficient to carry the depreciation and fixed charges, as well as the operation and maintenance expenses of the project. As applied to the reproduction method, this interpretation of going value is erroneous, because it deals with the "original" cost, and not the "reproduction" cost, of developing the present income of the plant; and because, while admitting that original cost is significant in a determination of the value of the corporation's property, the Courts hold that it is not of controlling importance, and have laid down the precept that the reproduction cost, as of the date of valuation or taking of the plant, shall be accurately determined as one of the standards of present value.

If such is the case with reference to the physical plant, why not with reference to the business of the corporation? Obviously, it should be so, for a moment's reflection will make clear that, with growth in population, the company has to face constant demands for new capital,

to meet the growing needs of the community, and this new capital passes through a period when it does not earn an adequate return, in exactly the same way as occurred in the case of the capital originally put into the plant. The going value of the plant, therefore, under normal conditions, should increase with the lapse of time, and with increasing investment in plant and business.

The Measure of Going Value.—What, then, is the measure of this going value, and what elements influence its amount?

Without going too minutely into a history of the development of the methods for the determination of going value, already fairly well established, suffice it to say that in a general way it has been found necessary to measure this element by a hypothetical comparison. This is done by determining the difference in earning capacity between the existing plant and a hypothetical new plant, which, for convenience, may be called "The Comparative Plant." This comparative plant is assumed to be built, beginning with the date of valuation, or taking, and to acquire business up to the level of the existing company as rapidly as possible. The sum of the present worths of the annual excess in net return of the existing plant over the hypothetical or comparative plant, in the period of years from the date of taking to the time when the earnings of the comparative plant are assumed to become identical with those of the existing plant, represents the going value of the existing plant.

The sum thus obtained represents the additional amount which a purchaser could afford to pay to the owners of the existing plant for its established income, over and above the cost of its physical property.

This method of arriving at going value may be termed the "Comparative Method."

The Investor at the Parting of the Ways.—In the case of a large plant, in which the period of years required for the reproduction or building of its physical property would be long, it will be helpful, in analyzing its going value, to place oneself in the position of an assumed capitalist, who stands, on the date of valuation, in the position of having in hand the necessary capital, so that he could either buy the existing going plant with its established business, or build a new hypothetical or comparative plant to replace the existing one under non-competitive conditions. In brief, this imaginary capitalist may be considered as standing at the parting of

the ways, with a choice of two paths, and as desiring to know from his engineer their relative financial desirability. This requires the engineer to determine for both cases the probable return which the capitalist would receive on his investment, during the period of years assumed as fairly necessary for a comparative plant to acquire the growing business of the existing plant.

Knowledge of operating conditions and growth of plants under different circumstances allows this comparison to be made with greater certainty than might be supposed. It is necessary, however, to be strictly judicial and thoroughly logical, in order to produce reasoning which will stand the test of cross-examination.

Some of this reasoning will be followed:

Should the capitalist take the first path, and buy the existing plant, he will evidently receive as return on his investment the net income indicated by its past history, modified by such circumstances as will be likely to influence its net income during the period when a new comparative plant can reasonably hope to overtake its development.

If he chooses the second path, that is, to build the new hypothetical, or comparative plant, he must be assumed to have in hand at the date of taking, a capital sum equal to the full reproduction cost of the existing plant and its business. This capital he must keep in such ready, or convertible, form as to enable him to meet the hypothetical costs and expenses as they arise.

Obviously, on this second path, as a builder of a new plant, he will not have to use the full amount of his capital at first, but will make only such expenditures as may conform to the reasonable construction requirements from year to year, until the end of the construction period of his comparative plant, and thereafter, such additional expenditures, up to the end of the going value development period, as may correspond to the cost of developing the business or going value of a newly starting company.

The capitalist then finds himself, in this second path, in the position of still having in hand at the end of the first year, a considerable proportion of his capital, on which portion he will be able to earn a rate of interest commensurate with the need of keeping it in convertible form. For comparative purposes, however, he will lose the return on the money expended during the first year for construction requirements.

Similarly, for the second, third, and additional years of construction and development of going value.

Clearly, if the existing plant is small, so that the construction period of a comparative plant will not exceed one year, the amount of unemployed capital on which interest can be earned, pending its use for construction purposes or acquirement of business, will be small, perhaps negligible, and will have little effect on the going value of the property in question.

In the case of a large plant, however, it becomes an element of importance, substantially reducing the amount of the going value which would obtain, were this return on unemployed capital neglected, and, therefore, it must be given careful consideration.

It remains, then, to compare these two paths in terms of returns on the two plants, through the period required for the development of the comparative plant. To do this, first find the differences in the return, under proper accounting methods, on the existing plant, in the light of its past history, and on the comparative plant, under reasonable assumptions as to development of income, including allowance for depreciation in both cases, but excluding consideration of the interest charges; then, to the sum of the present worths of these differences, add the sum of the present worths of the annual return on the unemployed capital. The result thus found should logically measure the going value.

It is evident that interest should not be included as an operating charge, for the reason that the relative return on the two plants, of which interest is a part, is the thing sought. To include interest or profit on capital (invested in plant, in contradistinction to unemployed capital) as an operating charge, would be to charge to operating expenses a portion of the very return which it is desired to value, instead of crediting it to the capitalist as a part of the return to be derived on the two comparative investments.

It is of importance at the beginning to understand clearly that all accounts with the capital under consideration must be begun at the date of taking, or at the parting of the ways, as it has been called, because the discussion of return is wholly a comparative one from this point of view. Some confusion has resulted, even on the part of those familiar with the subject, due to lack of appreciation of this fact.

The writers, therefore, will proceed to discuss more in detail this

question of return, by which is meant, in the case of the existing plant, the difference between gross annual income and operating, maintenance, and depreciation charges, excluding interest charges, however, and in the case of the comparative plant the same difference plus the income derived from the unemployed capital.

In discussing the development and operation of the comparative plant, the following considerations are of importance:

First.—The rate of interest for readily convertible capital is low. It will vary with the length of the period of construction, and to a lesser degree with the length of the period of acquisition of the business of the existing plant by the new comparative plant. For the bulk of the unemployed capital, held in readily convertible form, but awaiting use for construction purposes, 5% is too high a rate in most cases. Probably 4%, more or less, depending on local conditions, is a fairer rate. For that portion of the unemployed capital which is to be used for construction purposes during the year, unless its amount is very substantial, 2% a year, or thereabouts, on the average balance in bank, would seem to be a fair allowance.

Second.—Bankers will charge a discount, or brokerage, for the service of holding funds in readiness for use as required by construction needs, which will tend to decrease the net rate of return on the unemployed capital.

Third.—While this discount for keeping funds in readiness would reduce the average rate of interest, if the amount on deposit during the year is substantial, a portion of the bank balance can be kept in the form of certificates of deposit, on which a rate of interest higher than 2% should be earned. If the anticipated annual expenditures in plant construction are very large, a careful analysis may be made of the probable bank balances as affecting the rate of interest which might be allowed thereon by bankers. Under these assumptions, the average rate of interest which would be earned by the capitalist on his unemployed capital, less discount, would probably vary between 3½ and 4 per cent.

Fourth.—The larger the rate of interest allowed on this unemployed capital, the smaller the resulting going value of the existing plant. The actual effect (in percentage) on going value, is of lesser importance than would perhaps appear at first thought, for the reason

that the period during which there is unemployed capital on hand is small, as compared with the total going value development period.

In its more general aspect, the comparative method of computing going value may be said to conform to the actual experience of the investor in public service properties, and, to some extent, to the condition which he would have to face were he to build a rival plant, rather than to buy an existing plant. Such is the case when a city, desiring to duplicate a public utility, rather than to purchase the existing property, overlooks or willfully neglects the fundamental economic conditions which it must meet, assuming that a reproduction of the physical property is all that is necessary, without allowance for going value. Under such circumstances the property is reproduced under competitive conditions (contrary to the non-competitive fundamental hypothesis of the "comparative" method), which inevitably builds up the going value to large proportions. This has been the actual experience of certain cities such as Mobile, Ala., Sioux Falls, S. Dak., Newark, Ohio, and others.

The general theory of going value conforms strictly to the reproduction cost theory, and makes no assumption as to what the existing plant should earn on its investment, but is based on the actual past return derived by the existing plant and the probable return in the immediate future.

Furthermore, the results obtained by the application of this comparative method appear to comport with the range of going values, common in enterprises of this sort.

Principal Factors Affecting Going Value.

What, it may be asked, are the factors most affecting going value?

In the comparative method, aside from the relative gross earnings, and operating and maintenance expenses, the factors influencing most the magnitude of the going value are:

1. The period of construction required for building the assumed new hypothetical, or comparative plant;
2. The going value development period, or the period from the date of taking to the time at which it is assumed that the comparative plant may overtake the growing business of the existing plant;

3. The rate of acquisition of income by the comparative plant, during the going value development period; that is, from the date of taking to the time when the comparative plant is assumed to have reached the same stage of development of business as the existing plant;
4. The stage of development of the existing plant; that is, whether it may be said to be in a condition where its income is truly commensurate with the degree of development of its physical plant, instead of being either over-built or under-built;
5. The difference in annual depreciation charges assumed for the two plants;
6. The rate of return on the unemployed capital to be used in building the comparative plant, estimated as reasonable for capital which must be kept in a readily convertible form for application to construction.

Period of Construction.—The period and rate of construction of any comparative plant will be determined by the ability to plan and build wisely and economically. Experience has shown that, except in cases where the work is so scattered as to make possible its subdivision into a large number of contracts, the annual expenditure of a sum in excess of $1 000 000 is not an easy matter. On the other hand, it must be admitted that the desire to earn a return on the investment at the earliest possible moment, constitutes a strong incentive to push the work to completion as rapidly as possible, after its inception.

Order of Construction.—The order of construction will be influenced largely by the desire for the early development of income, and it is even reasonable to assume in some cases that temporary structures may be built, with this end in view, and abandoned after the completion of the permanent plant. The recent experience in building the water-works for the new city of Gary, Ind., furnished a striking example of this, for there some temporary pipe lines, temporary wells, and a temporary pumping plant were built to serve the city during the construction of the permanent water-works. These temporary structures were afterward abandoned, but they had served not only the economical purpose of making more convenient the work of the builders of this magic city, but also of reducing the cost of the going value of its water-works.

Going Value Development Period.—Obviously, the longer the period assumed as necessary for the development by the comparative plant of the growing business of the existing plant, the greater will be the going value of the existing plant. The effect of this factor is less marked than that of the rate of acquisition of income during this period, for the reasons that the differences in income decrease rapidly, and that the going value is not determined by taking the sum of the annual differences in net income, but of their present worths brought back to the date of taking or valuation, and the present worth factors become less and less as the period of years required for the development increases.

The past history of water-works, many of which were built during the Eighties, is not a fair criterion for the period required for the acquisition or development of business by water-works to-day, principally for the reason that the public is now educated to higher standards of living, and to the use of water under pressure, which would result in much more rapid development of income by water-works now than formerly. While some have urged that a water company is entitled to be credited with the cost involved in educating the public up to modern standards, was not this cost rather in the nature of an operating cost, than of a capital expenditure, inasmuch as this advantage immediately accrues, without cost, to any new-comer in the field, and cannot be held as a monopoly or asset by the existing company? It is usually assumed, therefore, that the public is educated to the use of water, and that the cost of that education is lost to the existing plant.

Rate of Acquisition of Income.—The rate of acquisition of business by a comparative plant is of the greatest importance, and especially so is the rate of development assumed for the first two or three years of its operation.

Several theories bearing on the rate of acquisition of the business of the existing plant in the comparative method have been advanced, the most important of which are as follows:

(1) The theory that as soon as a portion of the comparative plant is completed, the portion of the existing plant corresponding to it goes out of use.—Thus when the first stretch of distribution pipe system is laid, after the completion of the water supply system, the corresponding part of the existing company's system is assumed to be out of

service. The abutters on this pipe line have then the option of taking water from the new company, or of making other arrangements for obtaining their own supply.

Obviously, under such a drastic assumption, logical, but purely hypothetical, the rate of acquisition of business would depend substantially on the rate of construction of the plant, and, in the business section of the city at least, the income of the existing company would be derived by the comparative company very rapidly. In residential portions of the city, it is conceivable that the existence of wells and other facilities for obtaining water might result in a slower rate of acquisition of business.

(2) The theory that on the date of valuation, or taking, the existing company is notified that the city will build a new plant to replace the old, and that on the completion of the new system, the franchise and contract rights of the old company will be terminated.—Under such an assumption, the income of the old company would be much more gradually developed by the new company, than under the first theory, as many takers would continue to support the old company, as long as it was in business. Many of the old takers, however, would connect with the new company's mains and give up the old supply.

(3) The theory that the community now being served by the existing plant is without public water supply, and has never enjoyed the service of water-works.—Under such an assumption, substantially no service connections exist, and only those houses have water-works and plumbing fixtures as are assumed to have had their own individual water supplies. In substance, therefore, it is assumed that the comparative or starting plant is built in a new field, though the people of the community are assumed to be educated to the value of public water service under pressure, and may reasonably be expected to make connections with the mains of the comparative company and take water therefrom, as soon as they can prepare their houses for it. While this theory recognizes that the education of the public to the advantages of a public water supply has cost the existing company a substantial amount, it does not recognize that this cost should accrue to the company as an asset which it can hope to retain.

(4) The theory that the limiting rate for the acquisition of business by the comparative company is the speed with which service con-

nections can be made and pumping fixtures introduced.—This theory has little to commend it logically.

It will be observed that under none of these theories is the competitive theory developed; and with reason, for such an assumption would imply conditions entirely foreign to those under which the existing company, the going value of which is under determination, is operating, and would therefore not be a correct gauge or standard for determining its going value. Moreover, whether the danger of competition be eliminated by the character of the franchise, or whether it be prevented by the strength of the position of the existing company, or by the past experience of cities which have built competitive water-works, the water-works business in a large community is in fact virtually a controlled monopoly, and the best interests of the public dictate that this should be so. It might be added that control by an intelligent and honest public service commission is to be preferred, alike from the quasi-municipal corporation's point of view and that of the public, which it serves.

Momentary Stage of Development of Old Plant.—The going value of the existing plant will be somewhat affected by the relative stage of its financial development, or amount of its income, as compared with its momentary physical development. It must, therefore, be noted whether the existing plant is in what might be termed a "normal condition" or ratio of earnings to value of property, or whether it is in effect "over-built" or "under-built"; that is, whether, on the one hand, the company has recently incurred construction expenses in improving its facilities to which its income has not yet grown, or, on the other hand, has failed to make necessary improvements to keep the plant in first-class condition, thoroughly up to modern standards and the reasonable needs of the community which it serves.

Thus, one may recognize three conditions in which a plant may find itself:

(1) That of the "over-built plant," which finds itself in a position unable to earn operating and maintenance expenses, depreciation, and full interest charges and return upon its value, as a result of recent heavy capital or construction expenditures, unless perchance the return earned on the value of the plant prior to these construction expenditures was so unreasonable as to make the rate reasonable when the return is applied to the greater capitalization. In the final

analysis of value, weight should be given to this condition of the over-built plant, as well as to the past financial history of the company and existing conditions which may bear on the ability of the company to grow in income to the new capitalization; but it cannot be recognized in computing the cost of reproducing the income, or going value, of the existing plant.

(2) That of the "normally developed plant," in which the financial return is commensurate with the value of the property and the security or hazard of the business.

(3) That of the "under-built plant," the physical condition of which is obviously below a reasonable standard for the community which it serves, in which it is fair to assume that the existing net income will be substantially reduced by the expenses incident to the additional capital expenditures required to bring the plant to a rational standard of water-works construction or service. Here, again, the past history and present conditions, referred to above, are of importance.

Effect of Difference in Annual Depreciation Charges.—As regards the effect on going value of the relative annual depreciation estimated for the two plants, it is reasonable to assume depreciation to begin, in the case of the new comparative plant, on the completion and beginning of operation of its several component parts, which constitute workable units. In the case of a small plant, requiring not more than one year for construction, the contribution to the depreciation account or fund will not begin until the end of the first year after the completion of the construction of the entire comparative plant. In the case of a large plant—such, for instance, as that in which a period of construction of from 5 to 10 years would be required—it is reasonable to assume that contributions shall be made to a depreciation fund by the several portions of the plant as they are completed and put into operation; thus, assuming that a driven-well system, or pond intake, and a portion of the distribution pipe system had been completed as rapidly as possible, in the interest of developing an early income, depreciation should be charged to these portions of the plant on their completion and use, without waiting for the final completion of the entire plant, which might follow several years later.

The Rate of Return on Unemployed Capital.—The reasonable rate of return on the unemployed capital, which the builder of the comparative plant is assumed to have in hand, has already been discussed.

From $3\frac{1}{2}$ to 4%, more or less, was assumed to constitute a reasonable return, in the eastern part of the United States, on capital invested in sound, readily convertible securities, and 2%, or thereabouts, on full calendar months, on unemployed capital carried on deposit in the bank subject to draft at all times during the course of the year. The obvious effect of the magnitude of the rate of return on the unemployed capital is to reduce the going value of the existing plant. The higher the possible return on such convertible investment, the lower the going value.

Comparison of Conditions of Operation of Existing and Comparative Plants.

In making comparison between the operations of the existing and the comparative plants there must be considered:

(1) The relative income derived both from public and private sources;

(2) The relative operation and maintenance expenses; and, under this item, taxes and repairs are to be included, as well as the other items generally covered by the operation and maintenance accounts, omitting interest or return on capital invested in plant;

(3) The relative depreciation;

(4) The relative net revenue.

In determining the financial operations of the existing plant during the going value development period of the comparative plant, the past financial history and the conditions under which the plant is to operate must be analyzed carefully.

In determining the corresponding figures for the comparative plant, due weight must be given to the studies made of the operation of the existing plant, as well as of the conditions surrounding the comparative plant. The net reproduction cost estimate of the existing plant (after deducting accrued depreciation) will determine the construction cost to be assumed for the comparative plant.

Limit of Capital Account Charge for Interest During Construction.

In the construction cost of the comparative plant, as of the existing plant, there must be included an allowance for loss of interest during

construction quite independent from the interest "return" which will be earned later on the property of the company.

This apparent duplication of interest has been the source of some confusion in the minds of those who approach this subject for the first time. It will clear up the seeming paradox, however, to remember that interest-during-construction payments are inevitable in all projects; they cannot be avoided; they constitute a forced expenditure, alike with the cost of the bricks and mortar, the pipes and the pumps, and cannot be recovered in any way except by addition to capital account or floating debt. Interest during construction, therefore, is logically not a "return" or income in any such sense as one speaks of return, in comparisons for determining going value, but essentially a construction cost. The confusion has resulted only because the word "interest" has a different significance in the two cases.

The dividing line between interest chargeable to the item "interest loss during construction," which is added to construction cost, and "interest" which is return on the investment of the company, is drawn at the point where the several parts or units, making up the plant as a whole, are completed and put into operation, and thus begin to earn some income or return, instead of being a dead load to the company. On the other hand, it must be borne in mind that the plant cannot be expected to earn the full amount of the fixed charges, in addition to operating, maintenance, and depreciation charges, immediately on beginning the operation of the several parts of the plant, and that the losses during such formative period must therefore be carried to capital account, through going value, in precisely the same way that interest during construction must be carried to construction account. There is no other means of compensating the builder or investor for these early losses.

It would be as unfair to the owners of the public service corporation to fail to permit them to recover these early losses, as to make the public pay, through the agency of the rates, an annual return forever after the completion of the works, on excessive, wasteful, and foolish construction.

It is clear, therefore, that, after operation has begun, the early losses in interest on the investment or value of the plant form a considerable part of the going value, and that loss of interest during construction, up to the point of operation, must be credited to capital account as a construction expense.

Application of Principles Outlined.

Assumptions as to Existing Plant.—For the sake of illustration, and to make clear the principles which have been discussed, the writers have assumed a typical case in order to show the steps necessary to determine the going value of a large property on the comparative method herein outlined.

ASSUMED ANNUAL GROSS INCOME, OPERATING EXPENSES AND NET INCOME FOR EXISTING AND COMPARATIVE PLANTS 1910

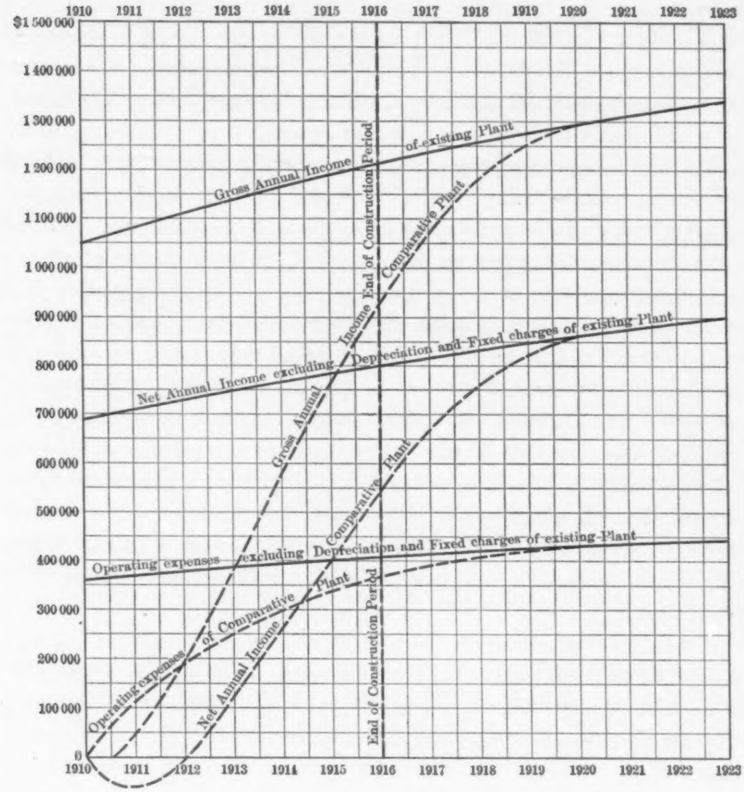


Fig. 1.

The plant which has been taken for illustration is an imaginary one, supposed to serve a population of 300 000 persons, but care has been taken to make the data consistent and conformable to figures found in operation. This statement is made, however, that the reader

may not be misled into supposing that the figures have been copied directly from the report of any public or private water-works.

In Fig. 1 is shown a forecast of the assumed probable gross annual income, net annual income (excluding depreciation and fixed charges), and operating expenses (excluding depreciation and fixed charges), of the assumed plant, called "the existing plant," which is supposed to be based on the past financial history of such plant, and the assumptions contained in Table 1.

TABLE 1.—General Statistics Relating to Assumed Water-Works Plant.
"The Existing Plant."

Year ending December 31st.	1910.	1915.	1920.
Population	300 000	330 000	365 000
Miles of pipe, all kinds	480	520	570
Population per mile of pipe	625	635	640
Taps in service (live)	40 000	45 000	51 000
Per 1 000 population	133	136	140
Per mile of pipe	83	86	88
Persons per tap	7.5	7.3	7.16
Consumption, annual, in millions of gallons	10 950	13 250	15 980
Daily, in millions of gallons	30	36.3	43.8
Per capita, in gallons per day	100	110	120
Per mile of pipe, in gallons per day	62 500	69 800	76 900
Per tap, in gallons per day	750	807	859
Hydrants (number)	3 840	4 420	5 130
Per 1 000 population	12.8	13.4	14
Per mile of pipe	8	8.5	9
Gross income (total)	$1 050 000	$1 190 000	$1 296 000
Per capita	3.50	3.61	3.55
Per mile of pipe	2 190	2 290	2 270
Per tap	26.25	26.40	25.40
Per million gallons consumption	95.90	89.90	81.10
Gross income, exclusive of hydrant rentals	$896 400	$1 013 200	$1 192 320
Per capita	2.99	3.07	3.26
Per mile of pipe	1 870	1 950	2 090
Per tap	22.40	22.50	23.40
Per million gallons	81.90	76.50	74.60
Cost of operation, exclusive of fixed charges and depreciation	$360 000	$402 000	$430 000
Per capita	1.20	1.22	1.18
Per mile of pipe	750	773	754
Per tap	9.00	8.94	8.44
Per million gallons	32.90	30.30	26.90
Percentage of gross income	34.3%	33.8%	33.2%
Net income, exclusive of fixed charges and depreciation	$690 000	$788 000	$866 000
Per capita	2.30	2.39	2.37
Per mile of pipe	1 440	1 510	1 520
Per tap	17.25	17.50	16.97
Per million gallons	63.00	59.40	54.20
Percentage of gross	65.7%	66.2%	66.8%
Hydrant rental (total)	$153 600	$176 800	$205 200
Per hydrant	40	40	40
Per mile of pipe	380	340	360
Taxes (annual)	84 000	95 200	103 680
Percentage of gross income	8%	8%	8%

Note.—The units have been computed by slide-rule.

It is further assumed that the investment in the physical part of the existing plant amounted to $8 600 000.

The problem, therefore, consists in determining: first, the interest loss during construction, and second, the going value.

Date of Valuation.—The valuation of this plant is assumed to be made as of January 1st, 1910.

Construction Period Required.—It is assumed that a period of 6 years—say from 1910 to 1916—would be reasonably required to build a new comparative plant identical with the existing plant, as of the date of taking in 1910.

Going Value Development Period.—It is assumed that a period of 10 years (or, in other words, 4 years in addition to the period of construction) will be required by the comparative plant to develop an income identical with that of the existing plant.

Beginning of Operation.—In spite of the fact that a 6-year period has been assumed to be required to build the comparative plant, it is assumed that a prudent investor would plan his order of construction so as to make available at the earliest possible moment certain parts of the comparative plant, such as the water supply and distribution pipe system, so that the latter could be put into commission and become a source of revenue at as early a date as practicable. Obviously, the magnitude of the interest charges on the comparative plant, during the construction period, would be a strong incentive to such a course of action.

Character of Plant and Order of Construction.—For purposes of illustration, it is assumed that the plant under question consists of a pond or driven-well supply, of limited extent, supplemented by a filtered river-water supply derived through large impounding or storage reservoirs, involving the construction, not only of a filter plant, but of expensive impounding reservoirs, dam, and conduits, two pumping stations, one to care for the small pond or driven-well supply, the other to care for the main supply from the impounding reservoirs.

The assumed order of construction is shown in Table 2, grouped under:

1. Distribution pipe system;
2. Pond or driven-well supply, and sterilizing plant, with pumping station;
3. Filter plant, conduits, etc.;

4. Pumping station for enlarged supply and extension of same after the year 1913 by the installation of additional pumping units;
5. Storage dam and impounding reservoirs.

The real estate, water rights, and rights of way are assumed to be distributed in the several divisions to which they appertain.

The cost of engineering and contingencies is also included under the several individual items cited above.

The interest during construction, however, as well as the going value, remains yet to be determined.

TABLE 2.—Interest During Construction and Progressive Amount of Investment.

"The Comparative Plant."

Date. (1)	Distribution pipe system. (2)	Pond or driven-well supply, sterilizing plant and pumping station. (3)	Filter plant, conduits, etc. (4)	Pumping station for enlarged supply and extension of same. (5)	Storage dam and impounding reservoir. (6)	Total Net Physical Value, Exclusive of Going Value and Interest During Construction. During year. (7)	To date. (8)
January 1st, 1910.	$400 000	$400 000	....	....	....	$800 000	
January 1st, 1911.	900 000	300 000	$800 000	$100 000	$100 000	1 700 000	$800 000
January 1st, 1912.	900 000	....	500 000	400 000	300 000	2 100 000	2 500 000
January 1st, 1913.	900 000	....	300 000	300 000	400 000	1 900 000	4 600 000
January 1st, 1914.	800 000	....	....	200 000	300 000	1 300 000	6 500 000
January 1st, 1915.	500 000	....	....	100 000	200 000	800 000	7 800 000
January 1st, 1916.							8 600 000
Totals........	$4 400 000	$700 000	$1 100 000	$1 100 000	$1 300 000	....	$8 600 000

Beginning of Operation of Different Portions of Comparative Plant.—It is assumed that by January 1st, 1912, the small supply from pond or driven-wells will be available. The filter plant and main pumping station is assumed to be completed and ready for service as of January 1st, 1914, the additional pumping units installed thereafter being ready for service between January 1st, 1915 and 1916. It is assumed that on or about January 1st, 1915, the storage dam and impounding reservoirs can be put into service, though not finally

completed until January 1st, 1916, and that the pipe distribution system built in any year is available for service the following year.

Loss of Interest During Construction.—In Table 3 has been computed the loss of interest during construction on this plant, based directly on the assumptions made as to the order of construction shown in Table 2 and the discussion following, and on the further assumption that the dividing line for the charges for loss of interest on construction account and on operating account is drawn at the point of completion and putting into service of any revenue-producing unit of the plant.

TABLE 3.—COMPARATIVE PLANT.
INTEREST DURING CONSTRUCTION.

Date.	Total Net Physical Value, Exclusive of Going Value and Interest During Construction.		Average amount on which interest is chargeable for one full year to capital account.	Interest on amounts in Column 4 at 6%.	Total Net Physical Value and Interest during Construction, Exclusive of Going Value.	
	During year.	To date, December 31st.			During year.	To date.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1910-1911	$800 000	$800 000	$400 000	$24 000	$824 000	$824 000
1912	1 700 000	2 500 000	1 650 000	99 000	1 799 000	2 623 000
1913	2 100 000	4 600 000	1 550 000	83 000	2 183 000	4 806 000
1914	1 900 000	6 500 000	2 650 000	159 000	2 059 000	6 865 000
1915	1 300 000	7 800 000	1 450 000	87 000	1 387 000	8 252 000
1916	800 000	8 600 000	500 000	30 000	830 000	9 082 000
Total allowance for lost interest-during-construction item				$482 000		

Thus, loss of interest during construction is allowed on the average amount of investment during the year 1910 at the rate of 6%, on the assumption that the work built during that year cannot be revenue producing until after the year 1911, pending completion of the water-supplying works. Similarly, interest is allowed during the year 1911, not only on the investment during the year 1910, but on the investment during the year 1911. After January 1st, 1912, however, the interest on the investment in pond or driven-well supply is assumed to be charged to operating cost as fixed charges, and not to capital account under the head of interest lost during construction, for the reason that it is assumed that this portion of the plant, being available through the agency of the pond or driven-well supply and the distribution

pipe system built up to that date, is now in the revenue-producing class. Similarly, interest lost during construction is charged to capital account on the filter plant, during the years 1912 and 1913, and thereafter to the operating account as fixed charges, for the reason that the filter plant is assumed to be in revenue-producing operation after January 1st, 1914, when it is assumed to have been completed, although the water-works plant as a whole is not assumed to be finished until January 1st, 1916.

New Construction in Going Value Development Period Ignored.— No consideration is taken of the new construction required during the construction and going value development period, 1910 to 1920, for the reason that, whatever profit or loss is involved thereby accrues alike to both plants in the comparative method, and so has no effect on the going value under determination.

Operations of Existing Plant.

In Table 4 are shown the assumed operating statistics for the existing plant.

TABLE 4.—Existing Plant.

Annual Income and Expense.

Year ending December 31st. (1)	Gross income. (2)	Operation, maintenance, and taxes. (3)	Net income, exclusive of depreciation and fixed charges. (4)	Depreciation. (5)	Net income applicable to fixed charges and dividends. (6)
1910..........	$1 050 000	$360 000	$690 000	$90 000	$600 000
1911..........	1 080 000	370 000	710 000	91 000	619 000
1912..........	1 110 000	380 000	730 000	92 000	638 000
1913..........	1 140 000	388 000	752 000	93 000	659 000
1914..........	1 165 000	395 000	770 000	94 000	676 000
1915..........	1 190 000	402 000	788 000	95 000	693 000
1916..........	1 213 000	409 000	804 000	96 000	708 000
1917..........	1 235 000	414 000	821 000	97 000	724 000
1918..........	1 255 000	420 000	835 000	98 000	737 000
1919..........	1 275 000	425 000	850 000	99 000	751 000
1920..........	1 296 000	430 000	866 000	100 000	766 000

Operations of Comparative Plant.—In Table 5 are shown the assumed financial operations of the new comparative hypothetical plant, without allowances for the income from unemployed capital.

Income on Unemployed Capital.—In Table 6 is shown the first trial computation relating to the income on unemployed capital. It will be remembered that it was suggested that, in this method of

computing going value, one should consider himself at the parting of the ways, with an amount of capital in hand sufficient to buy the existing plant—covering in the cost thereof the value of the physical plant, interest during construction, going value, and franchise value—and follow through the steps, relating to investment in the comparative plant, which the capitalist would take, and the return which he would get in the form of interest on his unemployed capital.

TABLE 5.—COMPARATIVE PLANT.

ANNUAL INCOME AND EXPENSE.

(From Operation, Exclusive of Interest on Unemployed Capital.)

Year ending December 31st.	Gross income.	Operation, maintenance, and taxes.	DEPRECIATION.		Net income applicable to fixed charges and dividends.
			Sum on which it is based.	Amount at 1%.	
(1)	(2)	(3)	(4)	(5)	(6)
1910	....	....	....	....	....
1911	$50 000	$118 000	....	....	$68 000
1912	195 000	195 000	....	....	0
1913	385 000	255 000	$2 123 000	$21 000	109 000
1914	575 000	305 000	3 106 000	31 000	239 000
1915	760 000	344 000	6 065 000	61 000	365 000
1916	923 000	372 000	7 152 000	71 000	480 000
1917	1 063 000	394 000	9 082 000	91 000	578 000
1918	1 175 000	410 000	9 082 000	98 000	667 000
1919	1 250 000	422 000	9 082 000	99 000	729 000
1920	1 296 000	430 000	9 082 000	100 000	766 000

Let it be assumed that he could place idle funds at a 4% rate, except as to the funds required for the year's construction, on which the bank rate of interest is assumed to be 2 per cent. For simplicity, let it be further assumed that on the money expended during one year, 6 months' interest at 2% can be obtained from the bank corresponding to a 2% rate on bank balances during the year.

Obviously, the first difficulty with which one is confronted is that the going value has not yet been computed. The comparative physical plant was assumed to have a value of $8 600 000; the loss of interest during construction was found, by Table 3, to amount to $482 000; the total investment in physical plant, including loss of interest during construction in the assumed existing plant and hence also in the comparative plant, is therefore $9 082. The going value, however, is not yet known, and, therefore, must be determined by a series of approximate computations. The franchise is assumed herein to have nominal value only.

TABLE 6.

First Trial Computation of Net Annual Income (from Operation and from Interest on Unemployed Capital) of Comparative Plant.

Assumed Capital in hands of the Investor-Builder on January 1st, 1910:

Value of physical plant..............................	$8 600 000
Interest during construction (Table 3).................	482 000
Assumed Going Value (First Trial)..................	1 800 000
	$10 882 000

Year ending December 31st.	Net income from operation (see Column 6, Table 5).	Determination of Interest on Unemployed Capital of Comparative Plant.								Total annual income or return on comparative plant. Column 2 plus Column 10.
		Total investment to date in comparative (physical) plant and interest during construction.	Going value.*	Total assumed investment to date. Column 3 plus Column 4.	Amount invested, which is earning 4% interest.	Amount withdrawn for construction during the year.†	One Year's Interest on		Total interest. Column 8 plus Column 9.	
							Column 6 at 4%.	Half of Column 7 at 2%.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1910....	0	$824 000	0	$824 000	$10 058 000	$824 000	$402 000	$8 000	$410 000	$410 000
1911....	$68 000	2 623 000	$179 000	2 802 000	8 080 000	1 978 000	323 000	20 000	343 000	275 000
1912....	0	4 806 000	485 000	5 291 000	5 591 000	2 489 000	224 000	25 000	249 000	249 000
1913....	109 000	6 865 000	811 000	7 676 000	3 206 000	2 385 000	128 000	24 000	152 000	261 000
1914....	239 000	8 252 000	1 126 000	9 378 000	1 504 000	1 702 000	60 000	17 000	77 000	318 000
1915....	365 000	9 082 000	1 395 000	10 477 000	405 000	1 099 000	16 000	11 000	27 000	392 000
1916....	480 000	9 082 000	1 607 000	10 689 000	193 000	212 000	8 000	2 000	10 000	490 000
1917....	578 000	9 082 000	1 752 000	10 834 000	48 000	145 000	2 000	1 000	3 000	581 000
1918....	667 000	9 082 000	1 842 000	10 924 000	— 42 000	90 000	— 2 000	1 060	— 1 000	666 000
1919....	729 000	9 082 000	1 884 000	10 966 000	— 84 000	42 000	— 3 000	1 000	— 2 000	727 000
1920....	766 000	9 082 000	1 897 000	10 979 000	— 97 000	13 000	— 4 000	0	— 4 000	762 000

* These items are determined most easily by carrying the computations involved in Tables 6 and 7 along simultaneously. It is thus assumed that the new plant develops its going value gradually.

† To cover physical plant, interest during construction, and going value.

As a first trial, assume a going value of $1 800 000. Under this assumption, the total value (or cost to the investor) of the existing plant would be as follows:

Physical plant............................	$8 600 000
Loss of interest during construction..........	482 000
Assumed going value......................	1 800 000
Total value	$10 882 000

Therefore, the investor who has concluded to build a new hypothetical plant, instead of to purchase the old plant, is assumed to have in hand the sum of $10 882 000, as of January 1st, 1910. He is assumed further, to adopt the same order of construction for the new comparative plant as shown in Table 2. Under these assumptions, Tables 6 and 7 result.

TABLE 7.—First Trial Going Value Computation as of January 1st, 1910.

Year ending December 31st.	Total Net Return.			Present Worth of Excess Earnings of Existing over Comparative Plant.			
	Existing plant.	Comparative plant.	Excess of existing plant.	Period of years.	Factor at 6%	Amounts.	Cumulative amounts.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1910.........	$600 000	$410 000	$190 000	1	0.9434	$179 000	$179 000
1911.........	619 000	275 000	344 000	2	0.8900	306 000	485 000
1912.........	638 000	249 000	389 000	3	0.8396	326 000	811 000
1913.........	659 000	261 000	398 000	4	0.7921	315 000	1 126 000
1914.........	676 000	316 000	360 000	5	0.7473	269 000	1 395 000
1915.........	693 000	392 000	301 000	6	0.7050	212 000	1 607 000
1916.........	708 000	490 000	218 000	7	0.6651	145 000	1 752 000
1927.........	724 000	581 000	143 000	8	0.6274	90 000	1 842 000
1918.........	737 000	666 000	71 000	9	0.5919	42 000	1 884 000
1919.........	751 000	727 000	24 000	10	0.5584	13 000	1 897 000
1920.........	766 000	762 000	4 000	11	....	2 000	1 899 000

Resulting Going Value..............................	$1 899 000
Assumed Going Value..................................	1 800 000
Error on First Trial Computation............	$99 000

Resulting Going Value under First Trial Computation.—In Table 7 is shown the result of the first trial computation of going value. It will be noted therein that the going value, which was assumed to

be $1 800 000, was found to be approximately $1 899 000, involving an error in assumption of $99 000.

Were a going value of $1 900 000 assumed in the second trial computation, the resulting going value would be found to be somewhat less than that sum, on account of the greater amount of interest accruing on the unemployed capital, which tends to reduce the going value, the other items remaining substantially the same.

Probably closer results would be obtained by assuming for the second trial computation a going value of $1 880 000.

Recomputation Unnecessary for the Purpose of This Paper.—As going through the somewhat laborious process of recomputing Tables 6 and 7 would not shed any additional light on the principles enunciated, it is unnecessary, for the purpose of this discussion, to follow through this recomputation. Enough has been said to illustrate the principles involved by the method of computing going value suggested by the writers.

For the second trial, assume a going value of $1 880 000. The total value of the plant under discussion would then be:

Physical plant	$8 600 000
Interest during construction	482 000
Assumed Going Value (second trial)	1 880 000
Total value	$10 962 000

In the second trial computation, therefore, it should be assumed that the builder of the comparative plant starts, as of January 1st, 1910, with the sum of $10 962 000 in hand.

Methods Identical when Franchise Value is Included.—Attention may be called to the fact that no allowance has been made for franchise value, on the assumption that the franchise has nominal value only, in the works under question, and that, under the forms of franchise enjoyed by many works, and with the present-day tendency of the Courts and public service commissions, water-works are approaching a condition where they may be limited in earnings to an amount which will pay dividends substantially only on the net physical and going value of the property. This has the effect of reducing franchise value to the mere right to do business.

If, on the other hand, the plant under question had been assumed

to have a substantial franchise value, as is the case in many works, this franchise value should have been added to the sum of the physical value, plus interest during construction, plus going value, to make up the amount of the unemployed capital which the investor is assumed to have in hand, before making his comparison between the existing works and a hypothetical works.

The preliminary costs of promotion, organization, etc., and other kindred items, are assumed to have been included in the physical valuation.

Effect of Assumptions as to Order of Construction on Loss of Interest during Construction Item.—It will be apparent, from the foregoing discussion, that the larger the allowance for the loss of interest during construction item, the smaller will be the resulting going value, for the reason that, the gross income and operating expenses remaining the same, the amount of unemployed capital in the hands of the builder, will be increased. This, in turn, increases the interest accretions on unemployed capital, which have to be deducted from the net income of the existing plant in determining going value. The effect, therefore, of these deductions is to decrease the going value. It is obvious, however, that the decrease in going value is not equal to the increase in interest charges, for the reason that the interest allowances on unemployed capital are assumed at a lower rate than the loss of interest during construction item, and the present worths of the annual differences in net income are taken in determining going value as of any given date, instead of the arithmetical sum of those differences.

Review of Going Value.

Having determined the going value of any works by the comparative method, it remains to review the matter in a broad way, for it must be remembered that cost is not always synonymous with value.

Thus, on the plant assumed herein, the resulting going value of $1 880 000 amounts to:

20.7% of its estimated net reproduction cost, meaning thereby the reproduction cost less accrued depreciation to date of taking, including the early organization expenses, engineering and contingencies, interest during construction, etc.;

17.1% of the total net reproduction cost, including in the latter the reproduction cost of the business of the company, or, in other words, the going value;

1.8 times the amount of the gross annual income of the old plant for the fiscal year ending January 1st, 1910;

3.1 times the amount of the net income, as of January 1st, 1910, meaning by "the net income" the difference between gross income and the sum of the operating expenses, maintenance, taxes, and depreciation.

It remains then to consider whether the plant is, at the moment:

(1) "Over-built"; that is, at the end of a reconstruction period, so far as the physical plant is concerned, and at the beginning of a recuperative period, so far as its financial condition is concerned, inasmuch as some of the construction work has been built for future rather than present needs, and the rates are therefore at the moment incommensurate with the character of the service being furnished;

(2) "In a normal stage of development" where the rates may be said, broadly speaking, to be commensurate with the character of the service furnished;

(3) "Under-built," that is, where the plant is outgrown, or where it is collecting rates which are excessive for the character of the service being furnished by it, in which case the plant may be said to be rapidly approaching a reconstruction or recuperative period physically and a reformative period financially.

Effect of Size of Plant.—It is obvious that in the case of a small plant, the construction of which can be executed in one season's work, no such complication in regard to the interest on unemployed capital nor necessity for refinement in computation exists.

The writers have felt it might be of interest, however, to call attention to the difference in the nature of the problem when dealing with a large plant requiring a period of several years for its construction, and to point out a method for the determination of the going value in such cases.

General Applicability of Principles.

The principles enunciated herein, if sound, should find general applicability in the valuation of all forms of business enterprises. Good will in general business enterprises should be susceptible of similar analysis to that herein outlined for public service corporations, when the theory of reproduction is utilized as the basis of appraisal, provided due weight be given to the modifying conditions or circumstances under which the property under valuation is operating.

Recapitulation.

Going value is defined as the value of a created income. While it pertains to the business rather than to the physical plant, it is nevertheless true that in any just determination of the reproduction cost of a property it is as real an element in this reproduction cost as is the cost of reproducing pipe lines, pumping stations, or any other part of the physical plant.

As a means of determining going value, the "comparative method" is suggested. This consists in an analysis of the relative net earnings or return to be derived from the existing plant as compared with that from a hypothetical "comparative plant" which is assumed to be built on the date of valuation and to acquire the business of the existing plant, in the territory served by it, as rapidly as possible, under non-competitive conditions. The sum of the present worths of the annual excess in net earnings, or return, from the existing plant, as compared with those from the comparative plant, in the period of years from the date of valuation to the time when the earnings of the comparative plant can reasonably be assumed to equal those of the existing plant, is then the measure of the going value of the existing plant. It represents the amount which a purchaser could afford to pay for the existing property with its established income, in excess of the value of its bare physical plant.

In analyzing the going value of any public service corporation, it is suggested that the investigator place himself in the position of an investor having in hand the necessary capital, either to buy the existing going plant with its established business, or to build a new comparative plant to replace it and its established business under non-competitive conditions, and then determine the return which he would receive upon his capital in both cases.

The principal factors affecting going value are:

1. The period of construction required for building the comparative plant;
2. The going value development period;
3. The rate of acquisition of income by the comparative plant;
4. The stage of development of the existing plant, whether overbuilt, normally developed, or under-built;
5. The difference in annual depreciation charges for the two plants during the going value development period;
6. The rate of return assumed upon the unemployed capital of the builder, up to the time of completion of the comparative plant.

The application of these principles is illustrated in the solution of a hypothetical problem, involving the determination of the going value of a water plant in a city assumed to have 300 000 population.

In the case of small plants, requiring but a year, more or less, for construction, all the refinements suggested in this paper are not necessary.

The principles outlined in the "comparative method" suggested, if sound, would seem to settle the controversy which has arisen as to the propriety of including interest or fixed charges as an operating expense. Clearly, if the principles enunciated are sound, the fixed charges should not be so included.

In conclusion, the writers dissent from the view taken by the Wisconsin Public Service Commission as to the nature and method of computing going value in determining the reproduction cost of a public service property.

Vol. XXXVII. FEBRUARY, 1911. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

SOME TESTS OF LARGE STEEL COLUMNS.

By James E. Howard, Esq.

To be Presented April 19th, 1911.

This paper describes the tests of five columns, representing members of Bridge No. 42, of the Pittsburg Division of the Pittsburg, Cleveland, Chicago, and St. Louis Railway, over the Ohio River, at Steubenville, Ohio. Through the courtesy of J. C. Bland, M. Am. Soc. C. E., Engineer of Bridges, Pennsylvania Lines West of Pittsburg, the writer is enabled to present the results of these tests.

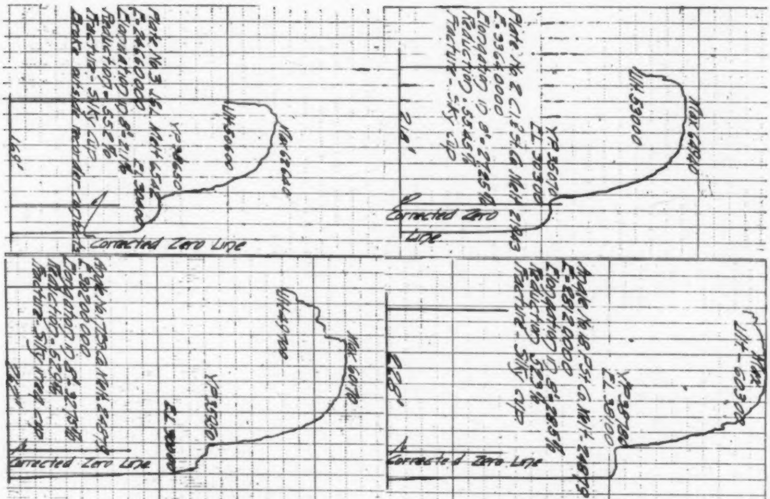
The tests were made at Phœnixville, Pa., excepting the preliminary loading of Column No. 1, which was done at Watertown Arsenal. The tests were conducted by the writer jointly with Mr. C. P. Buchanan, Assistant to Mr. Bland.

The outside cross-section dimensions of the columns were approximately 20 by 30 in., each having a sectional area of about 90 sq. in. The length of one column was 20 ft., and that of each of the others was 36 ft. 5 in. For test purposes, they were provided with 10-in. pin ends; in the bridge, however, the connections were all riveted.

The type of the columns and the details of construction are shown on Fig. 1. The columns were made of 6 by 6 by $\frac{5}{8}$-in. angles, with outside web-plates 30 in. wide and $\frac{11}{16}$ in. thick; and, between the legs of the angles, which faced inward, there were web-plates $17\frac{1}{2}$ in. wide

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

PLATE XV.
PAPERS, AM. SOC. C. E
FEBRUARY, 1911.
HOWARD ON
TESTS OF LARGE STEEL COLUMNS.



COMPRESSION TESTS, NO. 1.

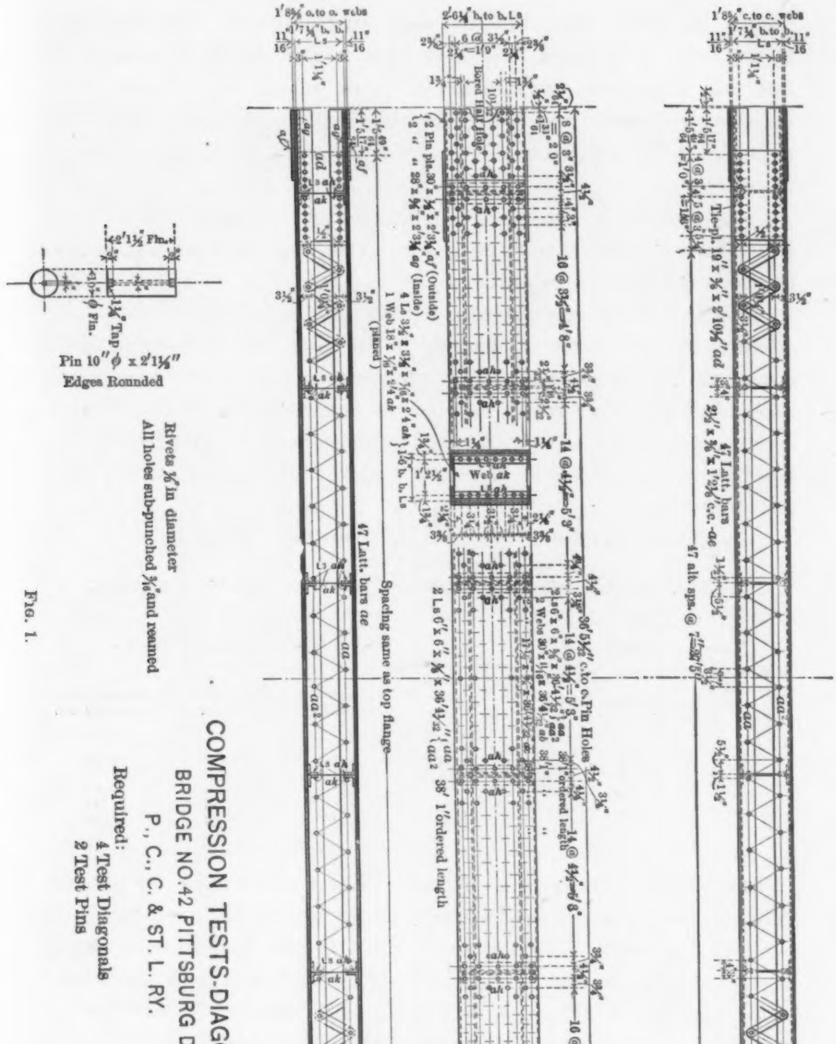

COMPRESSION TESTS-DIAGONALS
BRIDGE NO.42 PITTSBURG DIV.
P., C., C. & ST. L. RY.
Required:
4 Test Diagonals
2 Test Pins
Rivets ⅞ in diameter
Pin 10" φ x 2'1⅛"
Edges Rounded
47 Latt. bars ae
Spacing same as top flange
36' 5½" c. to c. Pin Holes
Web ak

FIG. 1.

and $\frac{5}{8}$ in. thick. The columns were single-latticed, with flat bars, $2\frac{1}{2}$ in. wide, $\frac{3}{8}$ in. thick, and 1 ft. $2\frac{1}{8}$ in. from center to center. At intervals there were diaphragms of $\frac{7}{16}$-in. plate. Steel rivets, $\frac{7}{8}$ in. in diameter, were used, and were driven in holes which were sub-punched $\frac{3}{16}$ in. and reamed $\frac{1}{16}$ in. larger than the nominal size of the rivets.

The ends were reinforced by pin-plates, the inside plates being $\frac{5}{8}$ in. thick and the outside ones $\frac{1}{2}$ in. thick, and each set was riveted to the web members with 39 rivets.

It will not be necessary to remark on the unusual opportunity afforded by these tests to acquire information on columns designed according to recent engineering practice, and in sizes representing work of magnitude. In addition to the determination of the ultimate strengths, observations were directed to matters of constructive details, and the behavior of the columns under loads preceding the final crippling ones.

The general results of the tests are shown in Table 1.

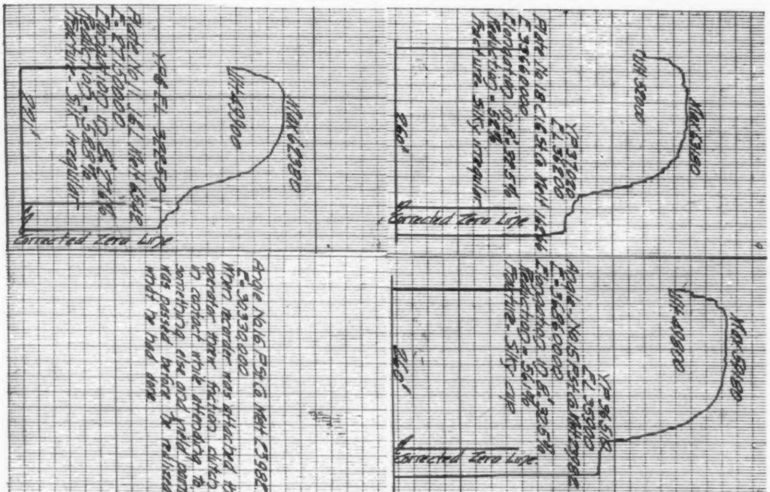
TABLE 1.—General Results of Tests of Bridge Columns.

No. of column.	Length.			Sectional area, in square inches.	Maximum Loads Applied.		
	Feet.	Inches.	$\frac{l}{r}$		Gauge reading.	Total, in pounds.	Pounds per square inch.
1	20	0	26.2	90.73	806	2 600 962	28 667
2	36	5	47.2	90.33	806	2 600 962	28 794
3	36	5	47.1	90.78	829	2 675 183	29 469
4	36	5	47.2	90.32	845	2 726 815	30 191
5	36	5	47.1	89.96	850	2 742 950	30 490

The ultimate strengths of Columns Nos. 1 and 2 were not reached. Columns Nos. 3 and 4 failed locally at the pin-plates, immediately in front of the pins. Column No. 5 failed by the web-plates buckling inward, at a distance of 12 ft. 8 in. from the end, causing a sharp bend in the member at this place, with a sidewise deflection, parallel to the axes of the pins. In the direction of failure the radius of gyration was 9.15 in., against 9.84 in. at right angles thereto.

Although, in the tests of two columns, the maximum stresses which were capable of being sustained by the columns were not applied, while two others failed locally, it is believed, nevertheless, from the indications of the tests, that the ultimate strengths of all were fairly represented in that of Column No. 5.

PLATE XVI.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1911.
HOWARD ON
TESTS OF LARGE STEEL COLUMNS.



COMPRESSION TESTS, NO. 2.

The grades of metal used in the fabrication of these columns are shown by Table 2, which gives the chemical analyses of the melts which furnished the plates and angles.

TABLE 2.—Chemical Analyses from Melts Furnishing Plates and Angles.

	Melt.	C.	Mn.	P.	S.
Plates	6 542	0.22	0.54	0.019	0.033
	16 266	0.27	0.51	0.007	0.034
	31 943	0.25	0.46	0.021	0.040
Angles	23 982	0.28	0.47	0.008	0.041
	24 879	0.18	0.53	0.007	0.032

Tests and analyses of rivet steel of the same grade as that used gave the results in Table 3.

TABLE 3.—Results of Tests and Analyses of Rivet Steel.

Melt.	Pounds per Square Inch.		Elong. in 8 in.	Contraction of area.	C.	Mn.	P.	S.
	Elastic limit.	Tensile strength.						
8 174	35 120	52 580	33	64	0.09	0.45	0.007	0.033
12 218	38 430	53 510	33	68	0.09	0.35	0.007	0.029
16 283	33 730	51 370	34	65	0.09	0.51	0.007	0.030
18 184	36 320	50 180	34	66	0.10	0.45	0.007	0.037

The quality of the metal of the plates and angles is further shown by the stress-strain diagrams, Plates XV to XIX, from the records of the Pittsburg Testing Laboratory. Tests pertaining to the same column are grouped on the same sheet.

Concerning the applicability of the results of tensile tests to the behavior of compression members, it may be remarked that the elastic limits in metal of this kind are generally coincident in value in tension and compression, and the characteristics of the stress-strain curves are similar for loads up to and in the vicinity of the elastic limits.

Considerable variation appears in the elastic limits of the tensile tests in metal from the same melt, and, furthermore, the plates and angles assembled in the same column were characterized by differences of this kind. That a column would be benefited by having the elastic limits of the component parts of substantially the same value, there

can hardly be a doubt, but, practically, a variation of some degree is to be expected.

Maximum and minimum values of the elastic limit and yield point found in the same melt are given in Table 4; and Table 5 shows the elastic limits of the principal component parts of each of the five columns.

TABLE 4.—MAXIMUM AND MINIMUM VALUES OF ELASTIC LIMIT AND YIELD POINT IN THE SAME MELT.

PLATE METAL.

Melt.	ELASTIC LIMITS, IN POUNDS PER SQUARE INCH.			YIELD POINTS, IN POUNDS PER SQUARE INCH.		
	Maximum.	Minimum.	Difference.	Maximum.	Minimum.	Difference.
6 542	32 250	28 370	3 880	38 650	30 940	7 710
16 266	36 200	29 600	6 600	39 200	33 460	5 740
21 943	31 300	30 300	1 000	35 070	34 440	630
METAL OF THE ANGLES.						
24 879	38 100	30 000	8 100	38 780	35 950	2 830
23 983	37 300	30 200	7 100	38 810	32 860	5 950

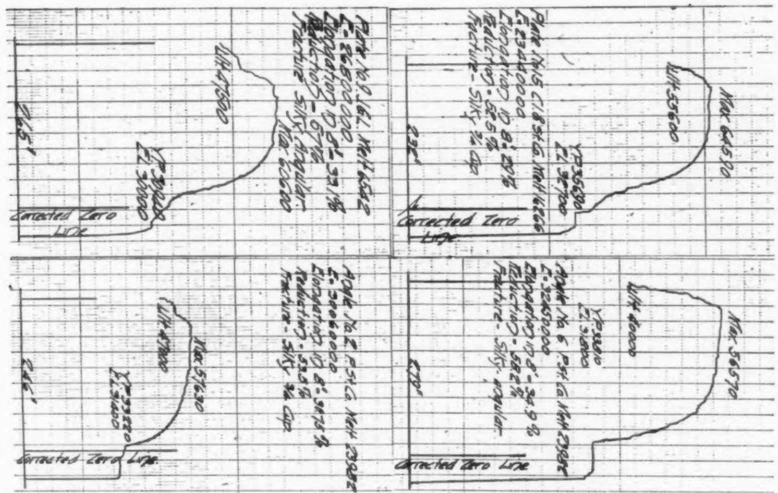
TABLE 5.—ELASTIC LIMITS OF THE METAL USED IN EACH OF THE COLUMNS.

No of column.	ELASTIC LIMITS, IN POUNDS PER SQUARE INCH.		No. of column.	ELASTIC LIMITS, IN POUNDS PER SQUARE INCH.	
	Plates.	Angles.		Plates.	Angles.
1	30 300*	38 100	4	33 200	34 500
	30 400	36 800		31 000	30 200*
	32 200	30 000*		28 370*	32 500
	30 400	35 500		31 000	32 600
2	36 200	35 900	5	34 600	36 900
	31 300*	35 100		29 600	33 800*
	32 250	32 300*		29 400*	35 060
	31 300	......		31 300	37 300
3	32 700	31 800			
	35 100	31 500*			
	30 000*	34 500			
	30 700	31 400			

*Minimum values.

The maximum and minimum values of the elastic limit, as a rule, were not found in those specimens in which corresponding values of the yield point appeared. As an index of ultimate compressive resist-

PLATE XVII.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1911.
HOWARD ON
TESTS OF LARGE STEEL COLUMNS.



COMPRESSION TESTS, No. 3.

ance, it is probable that the yield point is secondary in importance to the elastic limit, unless the two values coincide. It is believed that the minimum value of the elastic limit, as found in the component parts, chiefly modifies the ultimate resistance of the column, although it is probable that the shape of the stress-strain curve immediately beyond the elastic limit has an important influence.

If it is accepted that the elastic limit has a controlling influence on the ultimate strength of a column (as it is believed to have), then variations of from 25 to 27% in this value, as witnessed in the tests of the metal of these plates and angles, would overshadow those considerations which find expression in empirical formulas for strength, and take no account of such features.

The ultimate resistance of Column No. 5 rose about 1 000 lb. per sq. in. above the elastic limit of the weakest plate, but did not reach that having the next higher value. The reinforcing effect of the higher elastic-limit-metal of the angles and that of some of the plates was not adequate to raise materially the strength of the column. While a compact type of column might receive substantial benefit from its more rigid members, it would appear from the present example that such would not be the result ordinarily. It cannot be asserted, by reason of lack of identity, that the weakest plate in this column actually was the one which failed first, but it seems reasonable to assume that it was.

The detailed information developed by the tests has been entered on a series of photographic prints, the entries being placed on those parts of the column to which they refer.

Fig. 1, Plate XX, shows the longitudinal compression observed on gauged lengths established on different elements of Column No. 1. The compressive strains shown on this and subsequent prints were those due to a range in stress of 780 000 lb., between an initial load of 20 000 lb. and 800 000 lb., the maximum load applied at this stage. This range is equal to 8 597 lb. per sq. in., the maximum stress being equal to 8 817 lb. per sq. in. The maximum load, 800 000 lb., was applied several times before making the observations here recorded, which exhausted the early but slight permanent sets of the column, if there were any.

The compression of the web-plate of Column No. 1, on a gauged length of 150 in. established along the middle of its width, was

0.0408 in. At the edges of this plate the strains were 0.0394 and 0.0410 in., respectively, the mean of which is 0.0006 in. less than the observed strain on the middle gauged length.

Corresponding measurements on the edges of the web-plate on the opposite side of the column gave readings of 0.0467 and 0.0485 in., respectively.

The mean compression of the four corners was 0.0439 in., giving an apparent modulus of elasticity of 29 375 000 lb. per sq. in. This value is not far below that frequently assumed for steel, and therefore indicates a favorable action of the column taken as a whole.

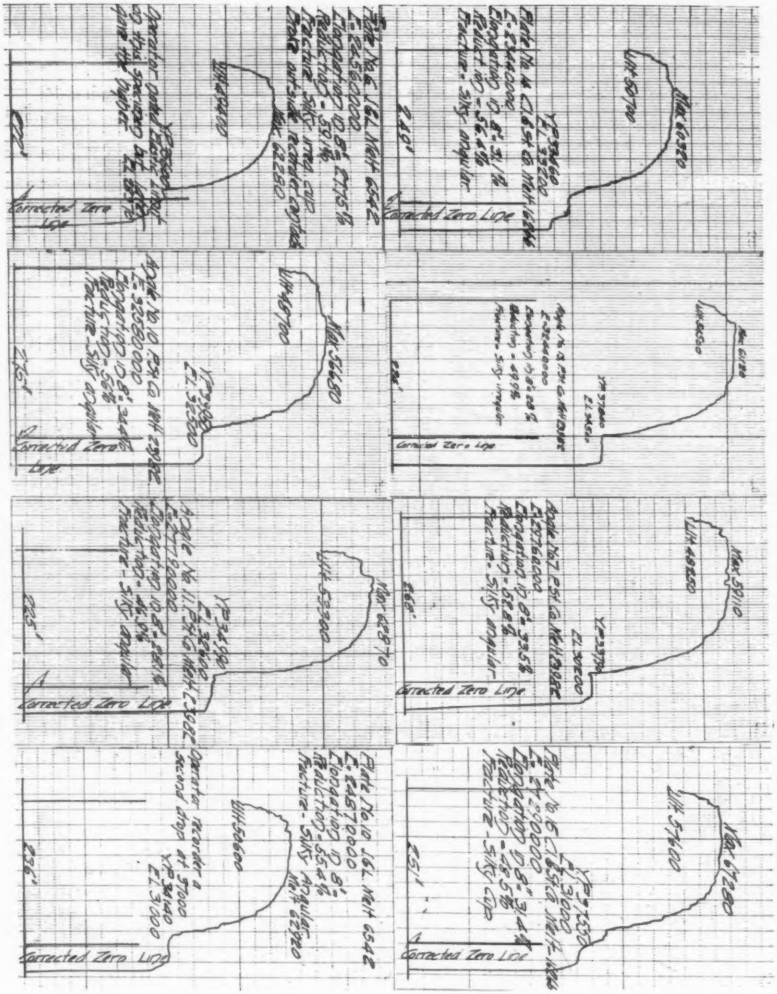
The rate of compression on individual gauged lengths was nearly uniform, while the stress-strain curves for ascending and descending loads followed closely the same path. Table 6 shows the compressive strains which were measured on the center line of the web-plate, as the loads were successively advanced and released.

TABLE 6.—Compressive Strains.

Total applied loads, in pounds.	Strains under:		Difference.*
	Ascending loads.	Descending loads.	
20 000	0.0003	0.0005	0.0002
100 000	0.0030	0.0032	0.0002
200 000	0.0078	0.0081	0.0003
300 000	0.0131	0.0137	0.0006
400 000	0.0187	0.0191	0.0004
500 000	0.0241	0.0246	0.0005
600 000	0.0296	0.0300	0.0004
700 000	0.0351	0.0355	0.0004
800 000	0.0410	......	......

* The difference represents a lag in resilience, in each case.

The compression of the column was measured along the center line of the web on two other gauged lengths, differing in their total lengths by 1 in., the longer one, 184.75 in., reaching from pin-plate to pin-plate, and the shorter length, 183.75 in., being taken wholly on the web-plate. The compressive strain from pin-plate to pin-plate was 0.0020 in. greater than that observed on the web. This strain, 0.0020 in., if it occurred on the solid metal, would correspond to a stress of about 60 000 lb. per sq. in., but the mean stress on the column was only 8 597 lb. per sq. in. In part, therefore, this strain signified an elastic movement of some other kind than a direct compression of the steel.



COMPRESSION TESTS, No. 4.

As the strains appeared and disappeared promptly when the loads were changed, it is inferred that there was no slipping of the pin-plates on the webs. None would be expected under as low a stress as that on the column at this time. The pin-plates were secured to the webs by the contractile force of 39 rivets, developing a probable gripping force of not less than $(0.6903 \times 39 \times 30\,000) = 807\,600$ lb. Considering that portion of the total load on the column which would be expected to come on the pin-plates, it would be thought that the riveting was fully adequate to prevent slipping, and that the compressive movement of 0.0020 in. represented combined compressive and shearing stresses, all of which were elastic.

This result, however, illustrates the constructive difficulty encountered in any endeavor to attain a state of rigidity in the reinforcing plates comparable to that existing in the continuous web members.

The measurements which next followed were on gauged lengths of 10 in. each, referring to local strains in the column, over the same range in stresses as before. These results are entered on Figs. 2, 3, and 4, Plate XX. On the north end, and also near the middle of the length of the column, five rows of gauged lengths were established along the center line, at the edges, and on intermediate lines, respectively. On and in the vicinity of the pin-plate at one end of the column twenty gauged lengths were established, as shown on Fig. 2, Plate XX. At the opposite end of the column, observations were made on the center line only.

The range of stress applied called for a compressive strain of 0.0029 in., taking the modulus of elasticity at 29 500 000 lb. per sq. in. On the web the observed strains were found to range from 0.0026 to 0.0033 in. The influence of the diaphragm and connecting angles, in modifying the strains, if any, could not be traced in the results.

In stepping down from the pin-plates to the web, greater strains were observed than those on the web member. Here the strains ranged from 0.0033 to 0.0043 in., confirming the excess movements previously observed on the gauged length which extended from pin-plate to pin-plate.

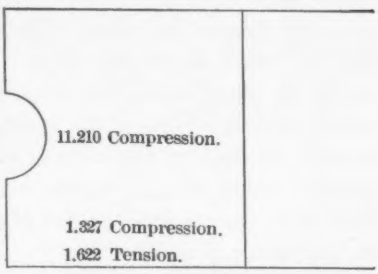
The compressions of the pin-plates ranged from a plus strain of 0.0040 in. to a minus strain of 0.0006 in. Directly in front of the pin the compression naturally reached a maximum, while abreast this place,

at the edges of the plate, the minus readings indicated that the metal there was in a state of tension. Diagonally inward from the pin, the metal was nearly in a neutral state, as regards resultant strains, the several readings being 0.0001, 0.0004, 0.0005, and 0.0003 in.

The compression on the bearing surface of the pin, when the load on the column was 800 000 lb., was 16 615 lb. per sq. in., using for a divisor the product of the diameter of the pin by the aggregate thickness of the metal of the web and the reinforcing plates.

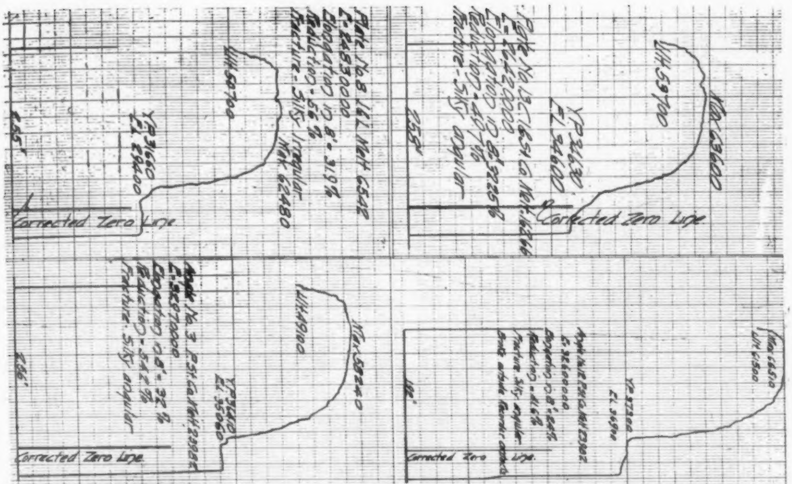
For the purpose of having approximate figures in mind, which may serve in some degree as an index of the action of the pin-plate, it may be stated that the measured compression of the metal in front of the pin corresponded to a mean stress of $\frac{29\,500\,000 \times 0.0038}{10} = 11\,210$ lb. per sq. in., while the corresponding stresses at the intermediate gauged lengths and at the edges were 1 327 and 1 622 lb. per sq. in., respectively, the latter being a stress of tension.

The compression of the tie-plate shown on Fig. 1, Plate XXI, was next measured. The longitudinal compression of this plate was that due to following and partaking of the compressive movement of the 6 by 6-in. angles, which in their turn followed the compression of the webs. It will be recognized that the efficiency of the riveting will be unfavorably studied in this plate, because of its attachment to a part of the column in which a complexity of conditions prevailed.



MEAN STRESSES, CORRESPONDING TO MEASURED STRAINS.
FIG. 2.

Measured near one of the lines of riveting, the compressions on three gauged lengths were 0.0008, 0.0012, and 0.0008 in. Checking these by one measurement over the aggregate length of 30 in. the reading obtained was 0.0030 in. The compression on the middle 10-in. gauged length is seen to be one and a half times that of the ends. This result is taken to signify that a progressive degree of compression occurs in the tie-plate, being a minimum at the end rivets and increasing with the number of rivets toward the middle of the length of the plate.



COMPRESSION TESTS, No. 5.

These results serve to direct attention to conditions which would be expected to prevail in a built-up member. The middle portion of a long plate, riveted to one of the principal members of the column, and measured on a longitudinal element, would reach a condition in which there would be uniform compression of the metal, practically. If the riveting did not reach to the end of the plate, some of the metal outside of but near the last rivet might be brought into a state of tension.

In a wide plate, the middle longitudinal element would be compressed less than the metal adjacent to the lines of rivets. This was exemplified in the present plate, where it was found that the compression along the middle of its width was 0.0011 in., against 0.0030 in. at the edge, both measured on a gauged length of 30 in.

The presence of tie-plates at intervals along the length of a compresssion member would, necessarily, relieve the continuous member of a portion of the load locally, the intensity of the stress varying from section to section according to the disposition of these secondary parts.

The metal in the different parts of the cross-section of the angles probably carried its proportional share of the total load. The results of two determinations, as shown on Fig. 2, Plate XXI, chanced to indicate a higher stress at the edge than that near the root of the angle. These results would have been reversed, one might think, if a substantial difference existed between the efficiency of the metal at the root and at the edges.

Observations were made on four of the lattice bars shown on Fig. 2, Plate XXI, but in none of them were any strains detected, the range of stresses applied to the column being as first mentioned.

Some results on the lateral expansion of the column appear on Fig. 2, Plate XXI, but detailed reference to them will be reserved until similar measurements entered on Figs. 3 and 4, Plate XXI, and Fig. 1, Plate XXII, have been described.

The ratio of lateral expansion to direct compression has been found to have the value, $\frac{1}{3.55}$. Using this value, the predicted lateral expansion of the web-plate, 30 in. wide, based on the mean compression of the two edges, as entered on Fig. 1, Plate XX, would be 0.0023 in. The observed lateral expansions ranged from 0.0019 to 0.0026 in. No modifications due to the transverse diaphragms were detected. On the pin-plates, notwithstanding the variations in direct compressive stresses which must have prevailed, the lateral expansions agreed closely, and

in one measurement coincided, with the predicted amount for the web. On the pin-plates, the high compressive stresses in front of the pins doubtless tended to modify the resultant lateral movements. This certainly was the case across the ends of the column, because each of those measurements showed minus lateral expansions of 0.0021 and 0.0025 in., respectively. These last readings, however, were consistent with the observations on direct compression, because the strains were in a tensile direction on certain of the 10-in. gauged lengths.

Turning back to the results entered on Fig. 2, Plate XXI, three transverse gauged lengths were established in one plane, one of which had its extremities on the angles on the lines of the lattice-bar rivets, one near the inner edges of the angles, and one near the corners of the column at the roots of the angles. As it had been ascertained that no strains were developed in the lattice bars by the loads of compression applied to the column, it would follow that the lateral expansion on the intermediate gauged length, which had its extremities on the line of the riveting, would be expected to correspond to the increased height of the triangle made by two of the lattice bars and the section of the angle between them, the angle being compressed its proportional amount of the total shortening of the column. It was found that the lateral expansion, measured on the intermediate gauged length, 0.0012, corresponded closely to that called for by the above assumption, the difference amounting to only about 0.0001 in.

The lateral expansion of the metal of the angle itself should furnish a correction to apply to each of the other gauged lengths, in order to arrive at the expected readings. Here, again, a close correspondence was found between the expected and the observed results. The correction, subtracted for the inner gauged length and added for the outer one, called for results differing from the observed expansions only about 0.0001 in. in each case. In other words, the widening of the legs of the angles and the change in height of the triangle formed by the lattice bars and the included section of the 6 by 6 in. angle accounted for the observed results, as would be expected.

An observation was made, from angle to angle opposite the diaphragm, shown on Fig. 2, Plate XXI. At this place the lateral expansion was 0.0001 in. as against 0.0012 in. on the corresponding length taken on the angles midway of the diaphragms. This difference in behavior was attributed to the restraining effect of the diaphragm.

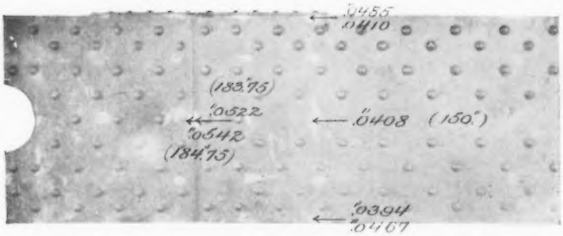


FIG. 1.—COMPRESSIVE STRAINS. COLUMN NO. 1, NORTH END. MEASURED ON GAUGED LENGTHS OF 150, 183.75, AND 184.75 IN.

FIG. 2.—COMPRESSIVE STRAINS. COLUMN NO. 1, NORTH END. MEASURED ON GAUGED LENGTHS OF 10 IN. EACH.

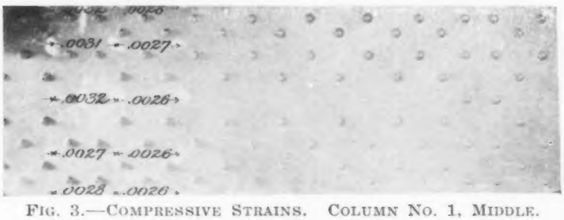


FIG. 3.—COMPRESSIVE STRAINS. COLUMN NO. 1, MIDDLE. MEASURED ON GAUGED LENGTHS OF 10 IN. EACH.

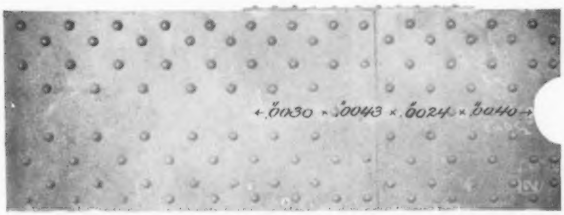


FIG. 4.—COMPRESSIVE STRAINS. COLUMN NO. 1, SOUTH END. MEASURED ON GAUGED LENGTHS OF 10 IN. EACH.

FIG. 1.—COMPRESSIVE STRAINS. COLUMN NO. 1, NORTH END.
MEASURED ON GAUGED LENGTHS OF 10 AND 30 IN., RESPECTIVELY.

FIG. 2.—STRAIN MEASUREMENTS. COLUMN NO. 1, MIDDLE.
ON ANGLES, LATTICE BARS, AND ACROSS COLUMN.

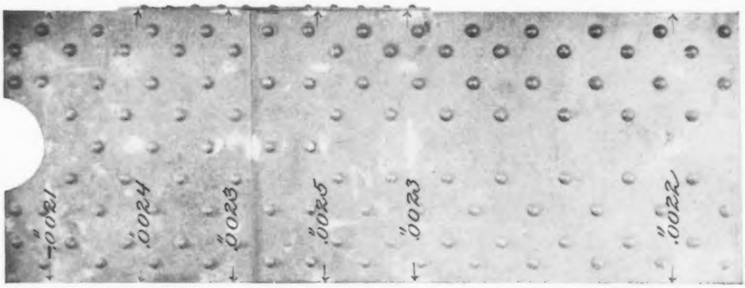


FIG. 3.—LATERAL EXPANSION MEASUREMENTS. COLUMN NO. 1, NORTH END.

FIG. 4.—LATERAL EXPANSION MEASUREMENTS. COLUMN NO. 1, MIDDLE.

This restraint might reasonably have led to the introduction of appreciable stresses in the lattice bars in that vicinity, but the connections were not sufficiently rigid, it would seem, for accomplishing that result.

When higher stresses were applied to the column, there was a slight compression of the lattice bar, shown on Fig. 2, Plate XXII, of about 0.0002 in. The column at this time was under a load of 28 667 lb. per sq. in. If the lattice bar was compressed the amount stated (some uncertainty pertained to this measurement), the corresponding stress was not more than 600 lb. per sq. in.

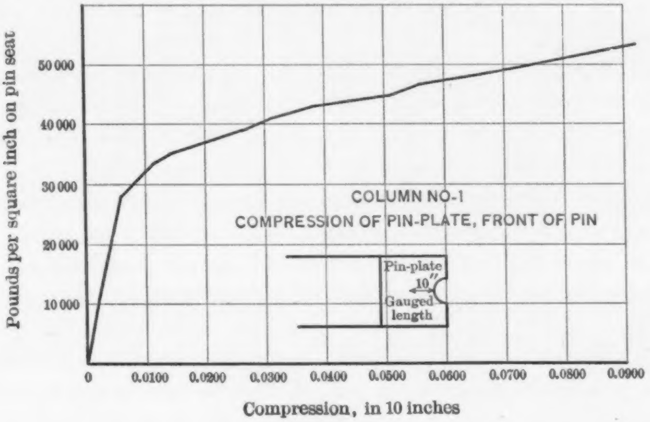


FIG. 3.

Further tests were made with this column, on the compression of the metal in front of one of the pins, using a gauged length of 10 in., as before. The results are plotted on Fig. 3. Rapid yielding occurred, as the load was increased from 28 000 to 33 000 lb. per sq. in., referring to the stress on the bearing surface of the pin. Although the stress on the sectional area of the column was then only from 15 000 to 18 000 lb. per sq. in., judging from its behavior in front of the pin, the ultimate resistance of the column could then have been predicted. It was apparent that the ultimate strengths of the columns would be found in the vicinity of 30 000 lb. per sq. in.

The measurements on this 10-in. gauged length, and subsequent ones on other columns of the series, were made with a strain gauge designed and used for the purpose of measuring strains in structural members under service conditions.

The disparity in sectional area and intensity of stress, between the body of the column and the pin bearings, directs attention and gives emphasis to a feature in this as well as in other compression tests, namely, that details of construction generally exert an influence on the behavior of the member, and not infrequently modify the ultimate resistance materially. Regularity of action is limited to stresses within, and ceases at, the elastic limit, and when over-straining of the metal occurs, locally or in general, the strength of the column is menaced.

The results next described refer to the permanent sets which were caused by the application of the maximum load to the column, and its release.

On Figs. 3 and 4, Plate XXII, are recorded the permanent sets, found on gauged lengths of 10 in. each, which resulted from applying a load of 28 794 lb. per sq. in. to Column No. 2. The compression on the bearing surfaces of the pins for this load was 53 353 lb. per sq. in. Necessarily, the greatest effect was produced in front of the pins. At one end of the column the permanent set was 0.1168 in., at the other end it was 0.1039 in. In other parts of the pin-plates the sets were comparatively of small magnitude, much less than found in the web-plates.

At the east end of the column the gauged lengths, which stepped down from the pin-plate to the web, showed the next largest movements to the metal directly in front of the pins. Evidently the pin-plate had moved bodily along the column in the direction of its length. At the opposite end the bodily movement of the pin-plate was not a pronounced feature of the test.

The mean set, per 10 in. of length, of the web of this column was 0.0030 in. The individual sets are seen to range from 0.0006 to 0.0180 in., showing an irregularity in the development of sets not unusual for stresses which are in the zone of, but just above, the elastic limit of the steel. In a built member the riveting may exaggerate the natural differences of the metal, and remarks in reference thereto will be made later in this paper.

The permanent sets of Column No. 3 were observed, on the center line, at each of the four corners, the results being entered on Figs. 1 and 2, Plate XXIII. The large set, measured on the north side and east end of the column, in stepping down from the pin-plate to the web,

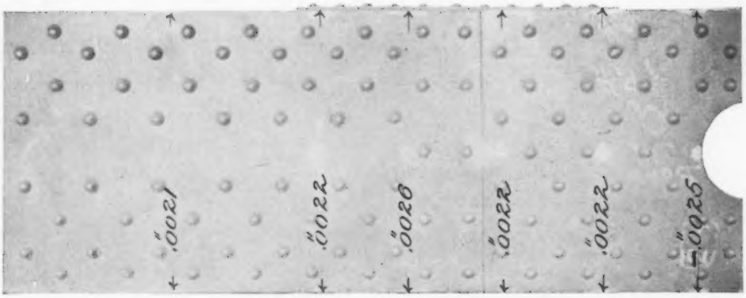


FIG. 1.—LATERAL EXPANSION MEASUREMENTS.
COLUMN NO. 1, SOUTH END.

FIG. 2.—STRAIN MEASUREMENT ON LATTICE BAR.
COLUMN NO. 1, EAST END.

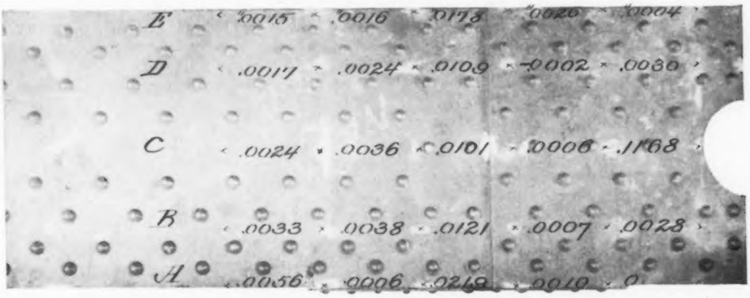


FIG. 3.—PERMANENT SETS. COLUMN NO. 2, EAST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

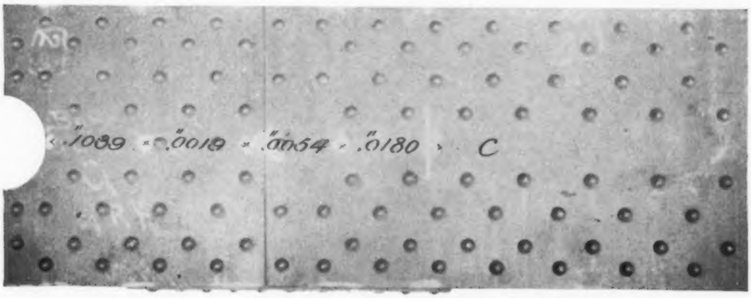


FIG. 4.—PERMANENT SETS. COLUMN NO. 2, WEST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

was due to a dishing of the plates in front of the pin causing a bend in the web at the end of the reinforcing plate. The local bending had the opposite effect on the south side of the column at this end, where the slipping of the pin-plate appeared to be zero.

The earliest set of the metal of the pin-plate naturally occurs immediately at the front of the pin, thence extending forward as the loads are increased. The affected metal in these columns was generally confined within the limits of the first 10 in., measuring from the pin. Column No. 3, however, afforded an example in which the sets reached into the second gauged length. At the west end the yielding of the metal at the pin was more nearly the same on each side of the column than witnessed at the east end, where local failure at the pin-plate occurred.

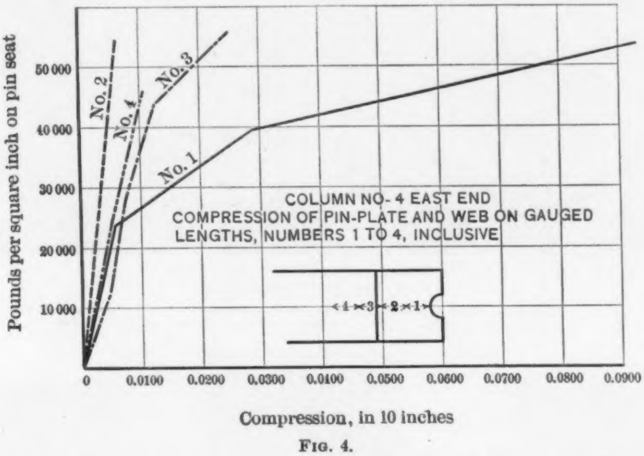


FIG. 4.

During the progress of the test of Column No. 4, the compressive strains developed on the reinforcing plates and the web, at each end of the column on the north side, were in succession measured by the strain gauge. These results are plotted on Figs. 4 and 5.

An inspection of the stress-strain curves of these diagrams shows the relative rigidity of the metal on each of the four gauged lengths. The metal least compressed was the part of the reinforcing plates covered by gauged lengths No. 2, a consistent result, it would seem.

Under the earlier loads, the greatest movements were observed on lengths No. 3, from which it appears that the reinforcing plates were carried along, lengthwise of the column, at an excessive rate when con-

sidering the movements observed on lengths No. 4. It is difficult to account for this excess movement, unless it is assumed that the pins were bent sensibly or that the riveting was inadequate to hold the plates firmly in place.

As the loads were advanced, the compressive strains directly in front of the pins soon gained the ascendency as regards magnitude, which of course was maintained for the remainder of the test, gauged lengths No. 3 standing next in the order of yielding.

Fig. 6 was prepared to show the measured compression of certain of the columns with reference to the compression pertaining to a modulus of elasticity of 29 500 000 lb. per sq. in. The stress-strain curves of

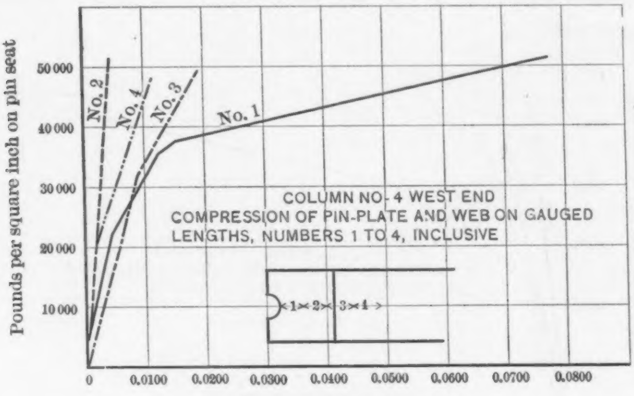


FIG. 5.

the columns are seen to follow closely the lines representing the above modulus, up to loads ranging from 12 000 to 20 000 lb.; the differences, if any, not showing on the scale used in the diagram. Under the higher loads, there is a gradual divergence, flattening the curve, and showing greater compressibility, which was probably due to the gradual development of permanent sets.

These curves represent the compressions on gauged lengths of 240 in. each, located on the web members. The extremities of the gauged lengths were nearly 100 in. from the ends of the columns, and, therefore, were quite beyond any local influence of the pins. The gradual development of sets seems to be characteristic of these built members, and is doubtless representative of columns of this type.

FIG. 1.—PERMANENT SETS. COLUMN NO. 3, EAST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

FIG. 2.—PERMANENT SETS. COLUMN NO. 3, WEST END.
ON GAUGED LENGTHS OF 10 IN. EACH.

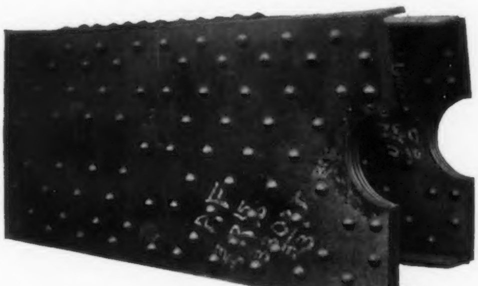


FIG. 3.—COLUMN NO. 3 AFTER TESTING. SOUTH SIDE, EAST END.

FIG. 4.—COLUMN NO. 3 AFTER TESTING. EAST END.

FIG. 1.—COLUMN NO. 4 AFTER TESTING.
SOUTH SIDE, WEST END.

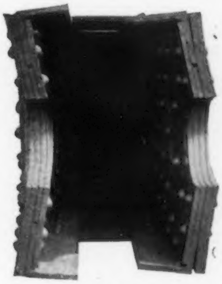


FIG. 2.—COLUMN NO. 4 AFTER TESTING.
WEST END.

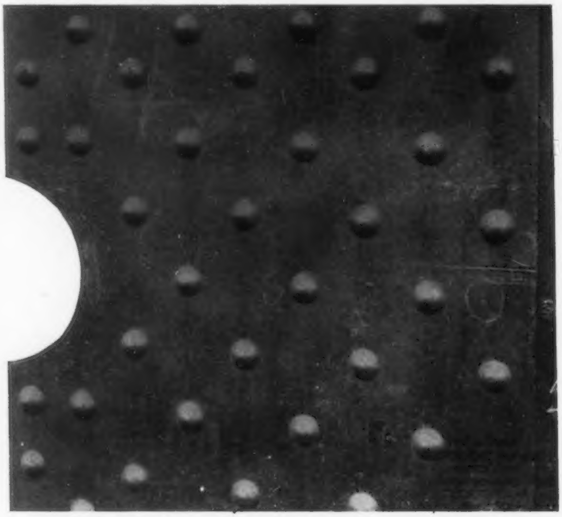


FIG. 3.—COLUMN NO. 4 AFTER TESTING.
NORTH SIDE, EAST END.

FIG. 4.—COLUMN NO. 5 AFTER TESTING.
SOUTH SIDE, AT PLACE OF FAILURE.

There are influences present in large built members which tend to modify their behavior, so that they do not act like plain steel bars. Plain bars of the grade of steel used in these columns would be expected to develop strains strictly proportional to the stresses in well-annealed, machine-finished specimens until the zone of the elastic limit was reached.

A jog in the compression stress-strain curve for loads just beyond the elastic limit would be looked for comparable to that displayed in the tensile tests. Such a jog sets a practical limit to the compressive strength of the material, excepting for very short pieces—for specimens

COMPRESSION STRESS-STRAIN CURVES OF COLUMNS WITH REFERENCE TO MODULUS OF ELASTICITY OF 29 500 000 LB. PER SQ. IN.

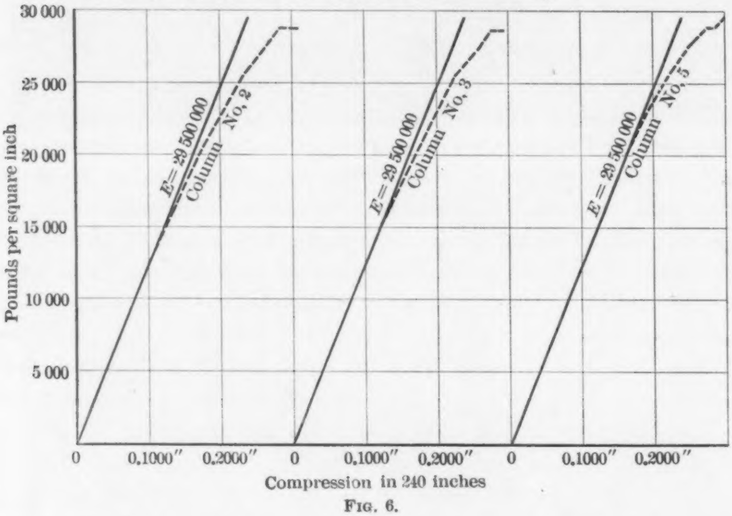


Fig. 6.

of the material rather than columns. In the case of a short compression specimen, frictional reinforcement, from contact with the platforms of the testing machine, raises the ultimate strength by reason of restricting lateral flow; and, provided there is symmetrical radial flow, no maximum resistance would be found. Such tests merge into those of cubic compression where, as far as known, no physical change results from the application of the highest stresses yet applied, and they have exceeded 100 000 lb. per sq. in.

It is known that, in steel and iron shapes, the parts first to cool after hot working are left finally in a state of initial compression. In

whatever direction the metal is tested the presence of initial strains tends to cause the progressive development of permanent sets, because of local over-straining of the metal under the combined effect of external forces and internal strains. The component parts of these columns were doubtless influenced by initial strains in some degree, which would cause the elastic limits to be vague and not sharply defined. Permanent sets, under these conditions, might be caused by stresses within the working loads, but confined to so small a part of the volume of the member as to render their presence negligible. It is, indeed, a matter of doubt whether any work of magnitude, involving the use of structural members of the kind under consideration, can be cited as an example in which no part thereof is strained beyond the elastic limit of the material.

The mean unit stress could be well within the elastic limit of the member, and yet local zones of overstrained metal might exist.

Initial strains, induced by the rapid cooling of the shapes from the temperature of rolling, may be augmented by initial strains introduced by mechanical operations on the cold metal. Straightening the shapes when cold, an essential operation, it would seem, introduces internal strains, and, unless the pieces are subsequently annealed, places them in a state of critical equilibrium, because of which they respond to applied stresses in part elastically and in part by the development of permanent sets.

Punching and shearing affect the metal over a wide area, as witnessed in the lines of scale which are disturbed about a punched hole or along the edge of a sheared sheet. The disturbance of the scale is evidence of the presence of internal strains. Reaming removes the most intensely strained metal at the immediate sides of the holes, but commonly does not remove all that was disturbed during the operation of punching.

At occasional intervals, internal strains may be introduced by the use of the drift-pin. When it is considered that the drifting of a 1-in. hole much beyond $\frac{1}{1000}$ in. over-strains the metal and introduces initial strains, it may be seen that any use whatever of the drift-pin is likely to be attended with the introduction of initial strains. The result may not be objectionable in a practical way necessarily, but such treatment contributes toward causing that divergence of the stress-strain curve in its upper portion from the right line representing a metal of no sets.

FIG. 1.—COLUMN NO. 5 AFTER TESTING. VIEW FROM WEST END.

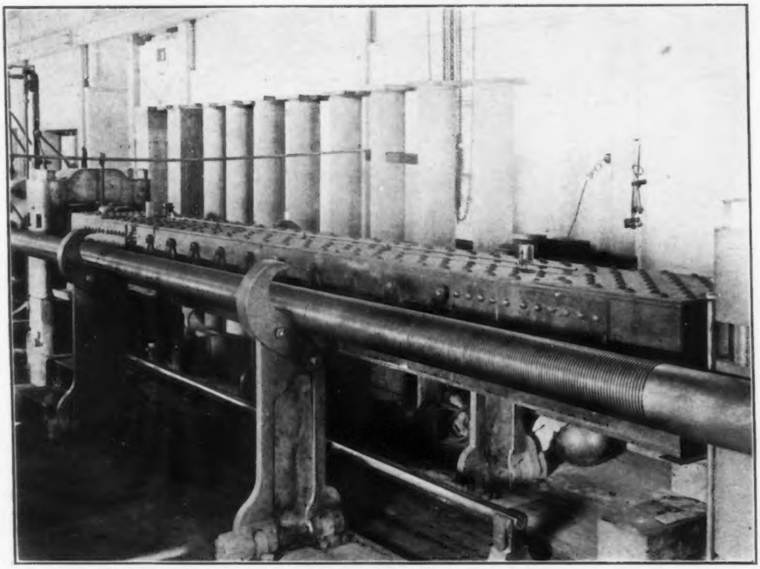


FIG. 2.—APPEARANCE OF COLUMN NO. 1 IN TESTING MACHINE.

The contraction of the rivets, in cooling after driving in the direction of their length, is also a cause for the introduction of internal strains. Rivets, in cooling down over a range of from 900 to 1 000° Fahr., contract some five or six times the distance which will extend them to their elastic limits when cold. Provided the plates which are being riveted are nearly flat, and lie close together, the contraction of the rivets should exert a pressure approximating the elastic limit of the rivet metal.

Assuming, however, that the contractile force is only 30 000 lb. per sq. in. on the stems of the rivets, the aggregate gripping force reaches a very formidable amount in a column of this type. On one of the longer columns of this series, the total gripping force of the rivets through the web-plates and lattice bars, on the above basis, reaches a total of more than 20 000 000 lb.

The section of the column within the limits of the gauged length of 240 in. included 456 rivets driven through the web-plates. The estimated contractile force on this part of the column exceeds 9 000 000 lb. The ratio of the annulus of the rivet head to the area of the stem is 1.35, from which it follows that a contractile force of 30 000 lb. per sq. in. on the stems would cause a mean compressive stress of 22 200 lb. per sq. in. under the heads.

From the above statement of the case, it would appear that the internal condition of the column is one characterized by the complexity of the strains which are present. The metal is not in a state of repose initially, that is, prior to receiving the loads of the testing machine, or such as would come on it when in its place as a structural member. The factors mentioned are believed to be adequate to account for the gradual flattening of the upper part of the stress-strain curves, as they were found and illustrated on Fig. 6.

Permanent yielding clearly took place for some time prior to reaching the maximum stress; and, logically, sets would be expected to appear first in localities where the internal strains augmented the effects of the loads applied in the testing machine. Scaling in the body of the column was first observed in the vicinity of the rivets, a very natural place, it would seem, for the metal to display the first signs of overstraining.

On a close approach to the ultimate resistance, time becomes a factor in the total amount of compression displayed by the column, the full compression not being reached immediately, as in the case

where no part of the metal is strained beyond its elastic limit. The loads on Columns Nos. 2, 3, and 5 were maintained for different periods of time, during which intervals the amount of compression increased without advance of load. The flattened tops of the plotted curves show where loads were held in this manner, the intervals of time being 1 hour 4 min., 2 hours 2 min., and 9 min., respectively.

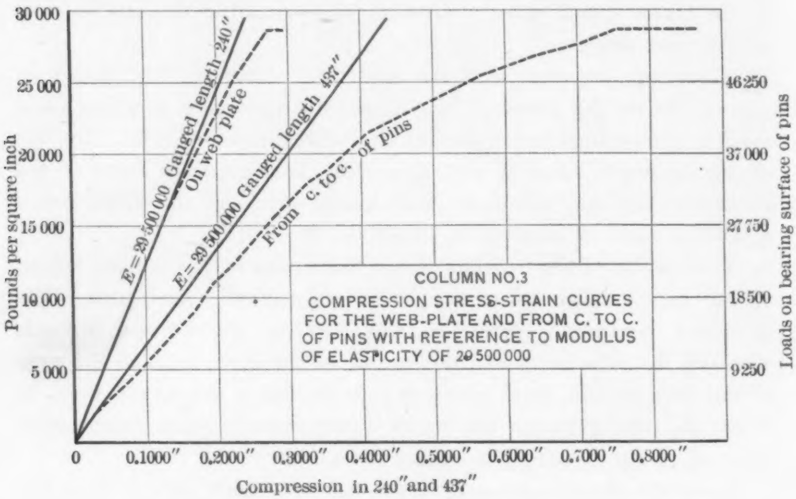


Fig. 7.

The apparent ultimate resistance of a column would doubtless be lowered by prolonged loading. In the present tests, however, the critical zone in which this result would be experienced is believed to have been but a narrow one.

Fig. 7 shows the relative compression of Column No. 3, when measured on the net section of the body and when measured from center to center of the pins.

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PAPERS AND DISCUSSIONS

THE VALUATION OF PUBLIC SERVICE CORPORATION PROPERTY.

Discussion.*

By Messrs. W. H. Williams, P. E. Green, E. Kuichling, Richard T. Dana, George T. Hammond, Leonard Metcalf, Charles Hansel, J. Martin Schreiber, Clinton S. Burns, Halbert P. Gillette, and Arthur L. Adams.

W. H. Williams, Esq. (by letter).—Before entering upon the discussion of the more essential elements of the problem presented by this paper, it seems worth while to correct one or two misapprehensions under which Mr. Riggs seems to labor, and to call attention to the rather extraordinary temper in which he approaches the grave questions with which he deals. Mr. Williams.

Mr. Riggs' first serious misapprehension is that railway officers, as a class, are, with substantial unanimity, opposed to any official valuation of railway properties, and that this opposition was voiced through the writer's discussion of Professor Henry C. Adams' paper in favor of valuation, at the last annual (December, 1909), meeting of the American Economic Association. Of course, on that occasion, the writer spoke, as he now speaks, only for himself, but, more than that, he then expressly disclaimed any such opposition, undertook to make suggestions as to the manner in which a proper valuation could be obtained, and directed his criticisms plainly at a proposal which contemplated, as he then observed:

"An incomplete and misleading valuation bearing the stamp and carrying the weight of governmental sanction, which can be of no practical advantage to the Government, the public, or the railways; but may easily injure the public and the railways by disturbing the confidence of the former and hampering the activities of the latter."

* Continued from January, 1911, *Proceedings*.

Mr. Williams.

The writer then added:

"It seems very clear that such a valuation as is proposed would be wholly useless to the Government for any practical purpose, because it would omit so many factors essential to any fair appraisement of the worth of the enterprises as going concerns."

Bearing in mind that the foregoing was addressed to the particular proposal made by Professor Adams, that being the topic on which the writer was invited to speak, a proposal expressly limited to the ascertainment of cost of reproduction less depreciation (the equivalent of cost of replacement with second-hand materials in a condition equivalent to that of the materials in use and hereinafter referred to as "cost of replacement") under the pseudonym of "physical value" (or sometimes "inventory value"), it would seem as though Mr. Riggs should sympathize with the writer's view, rather than with that of Professor Adams. Certainly, Mr. Riggs is fully aware of the inadequacy of mere cost of replacement to serve any useful purpose, for, after saying that:

"No account may be taken of the purpose for which the resultant figure of value is to be used; and the result should not vary, no matter what the purpose may be."

He says, in another place:

"* * * it is clear that the worth of the physical property, being the cost of reproduction less depreciation, is not necessarily the value of the property. * * *"

And, defining what he calls the "non-physical or intangible elements of value," says:

"These are those things which, added to or taken from the worth of the physical property, make up the value, and include whatever accrues to the property by reason of its operation, or by reason of grants, contract rights, competition, or location, which at the time of appraisal affect favorably or unfavorably the worth of the property."

The second misapprehension that is worthy of notice seems to have grown out of a curious sensitiveness, on the part of Mr. Riggs, as to any suggestion, other than his own, of criticism of any work undertaken or theories advanced by Professor Adams. As to every reader, other than Mr. Riggs, it is surely quite unnecessary to say that no attack has been made upon Professor Adams by the writer at the New York meeting of the American Economic Association or anywhere else. Certainly, it will be conceded that some difficulty would attend an effort to respond to an invitation to discuss before a scientific body a paper written by one of its members without making any allusion to the author of the paper or to his views or work, and those who have any knowledge of the history of official railway valuations in the

Mr. Williams.

United States, and especially of the proposal to undertake a Federal investigation of cost of replacement, are fully aware that Professor Adams has been from the beginning, and now is, the Hamlet of the drama, without whom it would become dull and lifeless. Strangely enough, Mr. Riggs seems to wish to deny to Professor Adams this prominence, for he says:

"Professor Adams was associated with the Michigan appraisal, but had no connection whatever with the 'physical valuation,' to which such objection is taken, and his appointment was made after the work of physical valuation had been fully outlined and was well under way."

It is true that the scheme devised by Professor Adams, and adopted at his suggestion by Governor Pingree, required the employment of civil engineers for the preliminary work which necessarily had to precede the final "valuation" by Professor Adams, but the bare statement of this fact is utterly misleading. Professor Adams' own testimony in one of the Michigan tax cases happily places his responsibility for the whole plan entirely beyond controversy. He said:

"In 1900 I was called upon by the Michigan State Tax Commission to determine whether railroads were paying a tax rate on their value equal to the rate on other property. With that problem in view, I formulated this inventory plan. * * *"*

Any discussion of the proposal for a National inquiry concerning cost of replacement which omits to show that its most persistent advocate, Professor Adams, has advocated and actually conducted or controlled several successive "valuations," in Michigan, as Statistician to the Interstate Commerce Commission, and as special employee of the Bureau of the Census, made in accordance with other methods than those which he now proposes to apply, is seriously inadequate; as seriously inadequate as it would be to omit to state that, using what purported to be the same method, Professor Adams, by changing the details of its application and decreasing the rates of interest used in his computations, raised his "valuation" of Michigan railways from \$152 958 202 to \$177 689 292 or 16.17%, each of the two calculations being presented to the public, with assurances that it disclosed the actual taxable value, and there being barely eighteen months between them. The writer is by no means alone as an object of Mr. Riggs' dissatisfaction because of public criticisms of Professor Adams' plan for estimating cost of replacement. Thus, of a statement in which Professor Taylor, who conducted the Wisconsin inquiry, questioned the validity of some of Professor Adams' methods, he writes:

"Undoubtedly this statement was made in good faith, and has gained currency by not having been corrected, but it is not the fact."

* Michigan Central *vs.* Powers Record, p. 500.

Mr. Williams.

In another place, referring to a statement of comparative costs to the respective States for valuation work, made by the Railroad Commission of the State of Washington, he says:

"It does not appear to be good taste either to criticize costs of work in other States, or compare the costs in Wisconsin and Michigan with the cost in Washington."

Referring to a paper by Charles Hansel, M. Am. Soc. C. E., who took part in the Michigan valuation, Mr. Riggs says:

"The one point to which special attention is drawn is Mr. Hansel's astonishing misconception of Professor Adams' plan of work. This misleading statement appears in the first paper and is reiterated in the second."

Again, of the report of the expert of the Washington Railroad Commission, who had the temerity to declare that it found "little value" either in Professor Adams' methods or his estimates of the cost of the work, Mr. Riggs says:

"Such sentences, and others which, by inference if not by name, reflect on work executed by men of high professional standing, are hardly in good taste, even if true, in a report to a railroad commission of another State."

Yet Mr. Riggs does not fail to criticize the method of "valuation," applied by Professor Adams in Michigan, in terms quite as definite as any used by others.

Thus, he condemns the method used to estimate the value of the non-physical elements appertaining to the Michigan railways, on the grounds (first) that it made this value a mere derivative of the rates existing, and (second) that it made no allowance for negative values when cost of replacement exceeded real value, saying:

"It will be seen that, in the case of a property in which the surplus earnings depend on excessive rates for service, it will fail as a method of determining a value for use as a basis of rate-making; and it fails, in the form in which it was used in 1900 and 1902, to bring out those negative or subtractive elements which may be determined from the income accounts, in the case of properties which do not earn a fair return on the investment."

Of the published statistics of American railways, compiled in the office of which Professor Adams is the responsible head, derived from annual reports made in accordance with forms prescribed by the Interstate Commerce Commission under his guidance, and containing items selected from and depending on the uniform railway accounting system devised by Professor Adams and imposed on the carriers by the Commission, Mr. Riggs writes:

"The published statistics are in such form that only the careful student of affairs can understand or analyze them, and but few of the public officials who receive them are able to read the reports of the properties and comprehend them."

Railway officers fall quite generally under Mr. Riggs' condemnation, for, of them he says: Mr. Williams.

"As a body * * * it is doubtful if any equal number of men, of equal intelligence, have as limited a knowledge of the fundamental truths of government, or knowledge so colored by bias. It is also doubtful whether any equal number of men have in their ranks so few who bear an active part in the duties and activities of citizenship, or who exercise large influence on their neighbors."

Such assertions as the foregoing need no comment; their intemperance is their most effective refutation; yet a few recent examples may be cited: Paul Morton resigned as Vice-President of the Atchison, Topeka and Santa Fé to become Secretary of the Navy in Mr. Roosevelt's cabinet; Jacob M. Dickinson, General Solicitor of the Illinois Central, became Mr. Taft's Secretary of War; his successor with the Illinois Central, William S. Kenyon, later became Special Assistant of the Attorney-General; Lloyd W. Bowers, General Solicitor of the Chicago and Northwestern, was Solicitor-General of the United States from early in Mr. Taft's administration until his death a few months ago. Thus, within but four or five years, the Federal Government took four of its highest officers from the railway officers located in only one of the country's great cities—Chicago.

Of a recent address by one of the ablest and most public-spirited of railway officers, he says:

"This address well expresses the spirit of the railway managers and employees toward all forms of investigation, and the complete lack of understanding, on the part of these managers, of the legal and moral relations which they bear to the communities which they serve."

Belonging to this so hateful class, and having also ventured to question whether Professor Adams has said the last and most perfect word on the subject of railway valuation, the writer is neither surprised nor disheartened to find that he, also, has caused Mr. Riggs undisguised dissatisfaction. It is a misfortune apparently inseparable from his profession and his conception of his obligations to his employers and to the public.

As has been already noted herein, the question is not whether railway property shall be officially "valued," but rather (first) as to how the "value" which is to be ascertained is properly to be defined, and (second) how the determination of "value," as properly defined, can be made most accurate.

The essential difference between the view advocated before the American Economic Association by Professor Adams and that of the writer was, and is, that the former now desires to exclude all elements of value which are not physical and tangible, while the writer holds that, if it is worth while to ascertain, on a general scale, at the cost of a necessarily large expenditure of taxpayers' money, and as to a

Mr. Williams. particular date, so unstable a fact as railway value, the kind of value the ascertainment of which could be of sufficient utility to warrant the effort can be nothing less significant than the "fair value" which the Courts have said is a proper element for consideration in fixing reasonable rates of charge. The fundamental difference between these two conceptions of value is admirably indicated by the following quotations, both of which rest on the authority of the Interstate Commerce Commission.

FAIR VALUE.

"The present value of a railroad property is necessarily very largely a matter of opinion only; it depends upon a vast number of contingencies and uncertanties, a road apparently of great value to-day may soon become worthless by the opening of a competing line having superior advantages or by the competitive struggles of other lines which operate to reduce the income of all; the value of a railroad largely results from the personal characteristics of its officials; the policy pursued by directors for the conservative and economical or progressive and daring, is a great factor in the determination of the current value of the property; a railroad property is not necessarily worth what it would cost to replace it and, on the other hand, it may be worth very much more than that."

Second Annual (1888) Report of the Interstate Commerce Commission, p. 64.

REPLACEMENT COST.

"The bill in question makes use of the phrase 'fair value.' Unless there is some legislative necessity, which we do not perceive, we question the advisability of using this phrase.

"It would seem to us preferable to substitute a phrase which indicates the fact that Congress desires an inventory valuation of railway property. By inventory valuation is meant that the property of the several railways shall be listed in detail, and that each kind or class of property so listed shall have assigned to it a valuation to be determined from the point of view of the contracting engineer, and not from the point of view of a court or board of arbitration which, from the nature of the case, cannot judge of what is 'fair value' except in the light of some specific use to be made of the valuation."

Letter of Hon. Martin A. Knapp, Chairman of the Interstate Commerce Commission, to Hon. Stephen B. Elkins, Chairman of the Senate Committee on Interstate Commerce, covering a then pending bill providing for railway valuation, March 25th, 1908.

As has already been noted herein, and amply verified by quotations, Mr. Riggs is fully aware that replacement cost and real value can rarely, if ever, coincide, and therefore plainly agrees, as to that elementary and essential point, with the writer and disagrees with Professor Adams, who would ignore or destroy every non-physical element of value in the property of all public service corporations. Mr. Riggs' recognition of the inadequacy of mere replacement cost is shown also by the excellent and convincing example which he cites*

* *Proceedings*, Am. Soc. C. E., for November, 1910, pages 1386-7.

of competitive railway routes between two Michigan cities which were built and are maintained and operated under such conditions that the far more costly of the two, which inferentially has correspondingly higher replacement cost, has much lower earning capacity, both as to gross and net, and is therefore actually worth much less than its less costly competitor. Mr. Riggs explicitly favors full recognition of the non-physical elements in every valuation; and, therefore, may be ranked as an opponent of any such scheme of valuation as that advocated by Professor Adams before the American Economic Association, or in the letter of the Chairman of the Interstate Commerce Commission, hereinbefore quoted. Mr. Williams.

Mr. Riggs, however, believes that the determination of the cost of replacement is an essential first step toward the ascertainment of real value. He says:

"The worth of the physical property is primarily that on which the value of the whole property rests."

The thought which the writer would place in opposition to the foregoing is that: Physical property has no value which is not an expression of its adaptation to economic needs. This is only another way of expressing the inevitable economic law, from which there is no escape, either in theory or in practice, that has been stated and sanctioned by the Supreme Court of the United States, as follows:

"But the value of property results from the use to which it is put, and varies with the profitableness of that use, present and prospective, actual and anticipated. There is no pecuniary value outside of that which results from such use."*

Mr. Riggs' own definition of value is not inconsistent with the foregoing. He says:

"The value of a property is its estimated worth at a given time, measured in money, taking into account all the elements which add to its usefulness or desirability as a business or profit-earning proposition."

The view of Mr. Riggs is that:

"While * * * the worth of the physical property, being the cost of reproduction less depreciation, is not necessarily the value of the property, * * * the physical worth must bear some very definite relation to value. * * *"

And he is, further:

"Strongly of the conviction that this relation is such that 'value' cannot be ascertained without a determination of physical worth."

It is exceedingly difficult to comprehend just what Mr. Riggs means when he describes the relation between real value (which he recognizes so clearly as value in use) and cost of replacement as "very definite."

* C., C., C. & St. L. Ry. *vs.* Backus, 154 U. S., 445.

Mr. Williams.

Certainly, he does not mean that it is a constant relation, or one which can be ascertained until there has been independent determination of both of the aggregates whose relation it expresses. In fact, the emphasis which Mr. Riggs places on replacement cost has led him into the grotesque fallacy of arguing that a correct estimate of real value is only to be attained by ascertaining: (first) cost of replacement, (second) real value, and (third) correcting the aggregate first obtained by applying whatever "very definite" relation (ratio) is necessary to make it agree with the second aggregate, which was from the beginning the only aggregate really wanted. The accuracy of this characterization of his proposed procedure is made perfectly clear by the following quotation:

"* * * the true method of valuing a corporate property is first to determine the cost of reproduction of the property and its depreciation, and modify this figure by any applicable positive or negative non-physical elements of value."

It is submitted that the clear meaning of the foregoing is that both replacement cost and real value as derived from use must be separately and independently ascertained, and that, these aggregates having been compared, the former is to be corrected by whatever allowance for non-physical value may be required to make it agree precisely with the latter. The obvious suggestion flowing from this discovery of his theory is that only value in use is wanted, as that is the only real value, and as it must be separately ascertained in any event, no other and *pseudo* value need be taken. The essential character of the method is as described, even when it is applied through determination of the annual value of the use and the assignment of one portion of such annual value to return on the capital value of the physical property and another portion to return on the capital value of non-physical property. The real nature of the method is not even effectually concealed by the capitalization of the income assigned to physical property at one rate and the income assigned to non-physical property at a different and higher rate. In fact, if it is necessary to conclude that a portion of the net annual income of railway property is normally paid to, or in respect of, a portion of capital entitled to a lower rate of return, and the remainder to or in respect of a remainder of capital entitled to a higher rate, the appraisal of the physical property is an excessively costly, cumbersome, and inaccurate expedient for determining the amount or value of either portion of the capital. Yet that is exactly what was done in Michigan by Professor Adams, the "valuation" he then made being completed before he altered his view by deciding that the non-physical elements of value are entitled to no consideration whatever, and that only cost of replacement is worthy of inclusion in an official "valuation."

But is there any real distinction between the "physical properties" and the "immaterial elements," such as the foregoing extract seems

to assume? Is not the superficial appearance of such a distinction plausible but deceptive? A locomotive is an entity; so is a railway. The separate parts of a locomotive are most of them independently valuable; so are the separate parts of a railway; but a large share of the value of the locomotive is the result of the nice adjustment of these separate parts to each other and to the work to be done. Mr. Williams.

Take a hundred different-sized locomotives, each adapted to different work under different conditions, and separate each piece of metal; it would be possible to value all these parts, but the aggregate would be far less than the value of the locomotives from which they were taken. Again, it would be possible to construct from these parts a hundred locomotives of such poor design, their respective parts so out of adjustment and balance, that they would be worth even less than the parts out of which they were assembled. The highest paid intelligence has not yet contrived the perfectly balanced locomotive, but a large part of the so-called "physical value" of every locomotive represents this sort of highly paid intelligence put forth at every stage from the opening of the mine where the ore was obtained to the delivery of the completed locomotive. Take ten railways of a thousand miles each, every one of them efficiently constructed, and equipped with proper terminals, stations, signals, rolling stock, and trained employees, and each properly adapted to the requirements of its territory and traffic; separate them into piles of ties and rails, groups of locomotives and cars, acres of land, unorganized bodies of men of varied capacity and training; what sort of intelligence will it require to build up out of these masses ten railways as efficient and useful as those that originally existed? Why, then, should the "physical value" of the locomotive include the assembling of its parts in proper balance and the "physical value" of the railway exclude the cost of the much more complicated adjustment of its elements of machinery and labor and location to each other?

At an early point in his discussion, Mr. Riggs makes an announcement, highly becoming on the part of one who proposes to deal with the problem solely from the point of view of a civil engineer, that he does not intend to argue the public utility of any sort of valuation, but only the method by which it may best be made, should one be determined upon. He says:

"This paper is confined to a discussion of the methods which should be used in arriving at a correct figure of cost of reproduction and depreciation—it does not take up questions involving the propriety of those figures when reached. The propriety or legality of using such figures as a basis for an assessed valuation, as a basis for rate-making (rate-making being an art in itself involving complications as great as those encountered in valuation), or any arguments as to the justice or injustice of legislation restricting issues of stocks or bonds, will be conceded no place in this paper. It is assumed that all these questions would have been taken up and a satisfactory answer reached before a valuation could have been ordered."

Mr. Williams.

Two pages after the foregoing paragraph, under the sub-heading "The Relation of Public Service, or Quasi-Public Corporations, to the People," Mr. Riggs proceeds to violate the wise, though self-imposed restriction, and devotes no less than eleven pages to a defense of the project on grounds of alleged public policy. In these pages he concludes that such a valuation as he proposes—not a mere determination of replacement costs, but a real valuation, with proper allowance for all elements of value in use—would be of service in connection with (*a*) taxation, (*b*) public control of rates, and (*c*) public control of issues of capital securities.

In supporting valuation as an expedient in taxation of railway property, Mr. Riggs seems to rely on a table made up from Professor Adams' Bulletin No. 21, as expert employed by the Federal Bureau of the Census, which table shows that the assessment of the railways of Wyoming for taxation purposes in 1904 was but 7.5% of their commercial valuation, as estimated by Professor Adams, and that this ratio varied greatly throughout the different States, running as high as 114.4 in Connecticut. Of course, nearly every one knows, even if Mr. Riggs does not, that the relation between the real value and the assessed value of all other kinds of property varies greatly from State to State, and even in different portions of the same State. On account of this variation, no table such as that offered by Mr. Riggs in support of his argument can have any value unless supplemented and explained by data covering the assessment of other kinds of property. It is worth noting, *en passant,* that the so-called "Commercial Valuation," on which Mr. Riggs rests this part of his argument, assigns a value equivalent to \$32 054 per mile to the railways of Michigan and one of \$45 211 per mile to the railways of the prairie State of Nebraska. Possibly this variation in the estimate of value is partly expressed in the conclusion that Michigan railways are assessed at 70.9% of their value and Nebraska railways at but 18.5 per cent. Obviously, there is no more need of uniformity among the States in the taxation of railway property than in their methods of deriving revenue from other kinds of property.

Also, Mr. Riggs admits that, when the Michigan valuation for taxation was made, it was not diminished, as it should have been, by the use of negative, non-physical value. This is fully equivalent to an admission that the method was unjust to every railway not capable of earning the full return on its replacement cost. He says:

"The use of a negative or subtractive non-physical value was considered, and advised by Professor Adams. * * *

"Professor Adams and his associates, therefore, applied only positive values, where any such were found, although advocating the use of negative values."

And, of the method then used, he says: Mr. Williams.

"* * * it fails, in the form in which it was used in 1900 and 1902, to bring out those negative or subtractive elements which may be determined from the income accounts, in the case of properties which do not earn a fair return on the investment."

And again:

"* * * where the earnings have been fairly uniform and stationary for a period of years, and the property does not earn a sufficient sum to care for depreciation and annuity, it is clear that the value as an earning investment is less than the determined physical value, and that the physical valuation should be reduced by some amount to arrive at the 'fair value.'"

In his argument favoring the use of a valuation in rate-making, Mr. Riggs affords no support to Professor Adams' contention that, for that purpose, only replacement cost should be considered, and that, after fixing the rates on the basis of the least favorably located and least efficient line, so as to afford it a bare return on its replacement cost, the surplus earnings at the same rates of its more favorably located or better operated competitors should be confiscated under the guise of a special tax. This extraordinary proposal, the character of which is so illuminating as to the attitude toward railway property and investments of the most prominent and persistent advocate of so-called "physical valuation," is best stated in Professor Adams' own words, which are as follows:

"I cannot evade the conclusion that equity, as between various classes of roads, can never be attained until all the excess of revenue over the Constitutional limit be made a contribution to the public treasury, and that this contribution be made as a substitute for all taxes of all kinds and all sorts."*

On the contrary, Mr. Riggs distinctly upholds the right to earnings in excess of the bare return, at the minimum rate of interest, upon the cost of replacement, saying, *inter alia:*

"It is contended that the determination of rates that will be just and fair to all competing companies involves other consideration than the valuation of either physical or intangible properties, and that when all these rate-making problems are properly solved, there will remain large intangible values on the well-designed plants."

Professor Adams has himself admitted that there is no possibility of utilizing any valuation for the purpose of fixing specific rates, as such a task is far beyond the capacity of any conceivable system of cost accounting. Supplementing this admission, Mr. Riggs' opposition to the plan proposed by the former and its gross injustice, so apparent to every one but its author, destroys the last element of plausibility in the suggestion that any sort of valuation could be of utility in that

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* *Proceedings* of the 22d Annual Meeting of the American Economic Association.

Mr. Williams.

connection. The writer is not overlooking the fact that the Courts, when under the necessity of repelling efforts to confiscate railway properties under the guise of rate regulation, and in view of the form in which this necessity has commonly presented itself, have accepted "fair value" as an element of importance in their inquiries; but if the railways are entitled to charge rates based on the value of the services they perform, it is clear that the question whether a rate or a schedule of rates is reasonably adjusted to the value of the service or services is very different from the question whether a fair return upon fair value has been allowed. Assuming, however, the need of an appraisement in every litigated case involving railway schedules, it is evident that each case would have to have its own appraisement, for value is ever changing and unstable. Mr. Riggs himself says:

"It is true that the 'value' of a property is an unstable figure, subject to fluctuations due to natural or artificial causes, and that a material change in value may occur suddenly, * * *"

Professor Adams proposed to keep his replacement cost up to date by annual accretions equal to annual expenditures for extensions and betterments; but this plan is illogical and inconsistent, for it proposes to ignore that very essential difference between original cost (less a proportionate allowance for wear and tear) and present worth, which is the very basis of the argument in favor of any valuation at all. Equally obvious objections, growing out of the instability of the ascertained value of any particular date, apply to any plan which does not provide for a re-appraisement every time the aggregate is to be used.

The objections to the use of any valuation for rate-making which have been cited are valid, and should be convincing, but they are insignificant by the side of the fundamental objection that, as Mr. Riggs says, "as a business proposition, the value of any property depends on its earnings," while those who would thus utilize a valuation are attempting to reverse the fact and make earnings depend on the value. Such a reversal is impossible. Ascertain real value and you have a consequence of earnings, past, present, and prospective, nothing else; use this as a basis for a rate schedule and you get, as a mathematical result, the present rates. The only way to derive any other result from this method would be to use as the basis some figure other than the real value, a method which would only be resorted to through moral turpitude or intellectual incapacity. One might almost assume that Mr. Riggs knows this, for he says:

"Value is given to a property, either by reason of the fact that it is an instrument for earning profit, or that it does earn profit or gives promise of profit."

The substance of Mr. Riggs' argument on capitalization control is that American railways are not often over-capitalized, but such evils do obtain in other industries, and therefore railway issues of capital

securities ought to be restricted.* Unfortunately, he gives no clue to the methods he would have applied, nor as to how far he would go in interference with the normal action and interaction of commercial forces in determining what securities can and ought to be issued. Railways are not over-capitalized. Table 9, a comparison of official valuations and capitalization, originally compiled by Mr. Slason Thompson, is instructive. Mr. Williams.

TABLE 9.

State.	Year.	Valuation by commission or tax board.	State proportion of capitalization.
Minnesota	1907	$411 735 194	$334 979 691
South Dakota	1909	106 494 503	108 911 000
Wisconsin	1909	284 066 000	249 299 060
Texas	1909	413 000 000	412 465 743
Washington	1908	186 007 490	153 493 940
Total		$1 401 303 187	$1 259 049 434
Excess of total valuation over total capitalization			$142 253 753

In view of frequent suggestions, in the public press and elsewhere, which indicate that there is a wide-spread opinion that the securities of railways have generally been watered, Table 10 is given. It is an analysis of the consolidated balance sheet as given in the reports of the Interstate Commerce Commission for 1908 and 1890.

Table 11 shows the length, in miles, of main and other tracks in 1908 and 1890.

The Commission, in its annual report, shows the securities issued per mile of road (first main track), but does not show the results per mile of main track (*i. e.,* 1st main track, 2d, 3d, 4th, and other main tracks), nor does it show the results per mile of all tracks (*i. e.,* main tracks, yard tracks, passing tracks, and industrial tracks). From the consolidated balance sheet, it will be noted that the securities per mile of road have increased 29%, while per mile of main track they have increased only 24%, and per mile of all tracks they have increased but 14 per cent. However, deducting the investments in stocks and bonds of other corporations, and showing the results only for the securities issued on account of the cost of road and 12% equipment, we have an average per mile of road of $62 388, an increase of 12%; and an average per mile of all main tracks of $56 166, an increase of 8%; and an average per mile of all tracks of $42 864, or a decrease of 0.7 per cent. It will be noted that a considerable part of these increases is due to increased cost of equipment, and the advantageous results obtained from such investment have been clearly shown. Of the invest-

* *Proceedings*, Am. Soc. C. E., for November, 1910, p. 1382.

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TABLE 10.—CONSOLIDATED BALANCE SHEET FOR RAILROADS OF THE UNITED STATES, EXCLUSIVE OF TERMINAL AND SWITCHING ROADS.

	Total.		Per mile of road.		Per mile of main tracks.		Per mile of all tracks.	
	1908.	1890.	1908.	1890.	1908.	1890.	1908.	1890.
ASSETS.								
RAILROAD:								
Cost of road	$12 035 195 403	$7 333 096 430	$56 268	$51 400	$50 656	$48 109	$38 659	$40 033
Cost of equipment	1 178 571 137	422 290 951	5 510	2 960	4 961	2 770	3 786	2 305
Material and supplies	226 250 462	63 785 950	1 058	447	952	419	727	348
Total	$13 440 017 002	$7 819 173 331	$62 836	$54 807	$56 569	$51 298	$43 172	$42 686
INVESTMENTS:								
Stocks owned	$2 115 313 379	$489 049 859	$9 890	$3 428	$8 903	$3 208	$6 795	$2 670
Bonds owned	1 271 311 512	241 115 665	5 944	1 690	5 351	1 582	4 083	1 316
Total	$3 386 624 891	$730 165 524	$15 834	$5 118	$14 254	$4 790	$10 878	$3 986
CURRENT ASSETS:								
Cash and current assets	$1 213 575 272	$307 874 188	$5 674	$2 158	$5 108	$2 020	$3 898	$1 681
Sinking, insurance, and other funds	154 975 409	125 095 987	724	877	652	820	498	683
Total	$1 368 550 681	$432 970 175	$6 398	$3 035	$5 760	$2 840	$4 396	$2 364
Miscellaneous	$1 277 458 795	$710 300 536	$5 973	$4 979	$5 377	$4 660	$4 103	$3 878
Grand total—All assets	$19 472 651 369	$9 692 609 566	$91 041	$67 939	$81 960	$63 588	$62 549	$52 914
LIABILITIES.								
SECURITIES ISSUED:								
Capital stock	$7 289 597 964	$4 179 156 990	$34 081	$29 293	$30 682	$27 417	$23 415	$22 815
Bonds	9 441 200 261	4 462 577 079	44 141	31 280	39 738	29 277	30 327	24 362
Total	$16 730 798 225	$8 641 734 069	$78 222	$60 573	$70 420	$56 694	$53 742	$47 177
CURRENT LIABILITIES:								
Accrued interest	....	$25 341 994	——	$177	....	$166	....	$138
Other current liabilities	$1 151 233 255	440 513 629	$5 382	3 088	$4 845	2 890	$3 698	2 405
Total	$1 151 233 255	$465 855 623	$5 382	$3 265	$4 845	$3 056	$3 698	$2 543
Miscellaneous	$845 115 552	$394 918 201	$3 951	$2 768	$3 557	$2 591	$2 715	$2 156
Grand total—All liabilities	$18 727 147 032	$9 502 507 893	$87 556	$66 606	$78 822	$62 341	$60 155	$51 876
Profit and loss balance	*745 504 337*	*190 101 673*	*3 485*	*1 333*	*3 138*	*1 247*	*2 394*	*1 038*
	$19 472 651 369	$9 692 609 566	$91 041	$67 939	$81 960	$63 588	$62 549	$52 914

ment in the track itself (cost of road), it will be noted that the cost per mile of main track has increased only 5%, while the cost per mile of all tracks shows a slight decrease in 1908 as compared with 1890. Mr. Williams.

TABLE 11.

Track.	1908.	1890.	Increase.	Percentage of increase.
Single track	213 888.36	142 665.89	71 222.47	49.9
Second track	20 209.05	8 437.65	11 771.40	139.5
Third track	2 081.16	760.88	1 320.28	173.5
Fourth track	1 408.99	561.81	847.18	150.8
Total, all main tracks	237 587.56	152 426.23	85 161.33	55.9
Yard track and sidings	73 728.57	30 750.17	42 978.40	139.8
Total mileage operated (all tracks)	311 316.13	183 176.40	128 139.73	69.9

"The Interstate Commerce Commission in 1908 report that their Balance Sheet covers 'miles of road' aggregating 213,888.36 miles, whereas their statement of mileage represents all roads reporting to the Commission whether or not they furnished a Balance Sheet.

"To analyze the Consolidated Balance Sheet, we have revised the statement of mileage to cover same roads as are included in the General Balance Sheet. The 'miles of road,' *i. e.*, miles of first main track, are actual. The Commission's report not showing separately for each line the miles of other main tracks or yard tracks and sidings, the figures shown in the statement of mileage are *approximate*. It includes mileage of all second, third and fourth tracks. Undoubtedly, practically all of the second tracks, third tracks and fourth tracks are owned, or operated by, roads furnishing the Commission with a Balance Sheet. Mileage of Yard Tracks and Sidings is based on the proportion which the single-track mileage of roads represented in the Balance Sheet bears to the total single-track mileage of roads reporting to the Commission."

These comparisons are more significant and convincing in the light of the large expenditures since 1890 for the reduction of grades, revision of line, interlocking towers, automatic block signals, increased weight of rail, increased capacity of bridges, improved stations and terminals, elevation of tracks, and the many other items going to make up the additions and betterments, and increasing the book cost of the property. The figures plainly prove that there has been no general practice on the part of the railroads of the country, from 1890 to date, of issuing capital securities without securing full value for the vast amount referred to. Why, then, should any restriction be placed on the form or manner of their future appeal for the very large volume of capital necessary to keep abreast of American industrial development? Why should they be limited as to what form of security they may offer in return for the cash capital which they must obtain if they are to serve the public adequately and properly?

It ought also to be borne in mind, in this connection, that, while there could be no lawful mode for the revision of existing capitalization, should it in any instance be found to be too small or too great when measured by the results of such a valuation, the future issue of securities must be controlled by the necessities of the carriers and the state of the market, and is also practically restricted by the Interstate Commerce Commission's accounting system, which declares what ex-

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Mr. Riggs considers *seriatim* nine objections to the ordinary methods of estimating cost of replacement which were mentioned specifically by the writer, as among the most important commonly omitted items, in an address before the New York Traffic Club, delivered during January, 1909. He concedes that the writer is correct in urging that allowances for "working capital with which to carry on the business" and for "impact and adaptation" ought to be included, and were omitted in Michigan and have been usually omitted. These are two of the nine objections specifically raised. As to five others, Mr. Riggs seems to be in considerable doubt. Concerning the objection that an allowance of 3% for interest during construction is too low, he contends that it was justified in Michigan by the "assumption," that the whole work of replacement would be accomplished in one year, and also "that on long roads partial operation would commence as various sections of the line were completed." He admits that these assumptions "clearly would not be proper" under different conditions, but appears to hold that they were warranted as to the Michigan work.

Another of the writer's objections was the absence of an allowance for "wear and tear of materials during the period of construction." As to this, Mr. Riggs says:

"This deterioration is a necessary incident to any construction work. It has not been customary or usual to take account of it. To add to the amount capitalized on account of this item would be manifestly improper. The only way in which this could be cared for would be in an adjustment of the depreciation reserve when raised to cover that which takes place during the construction period."

Of course, the depreciation account, when there is one, is a charge to operation. Therefore, Mr. Riggs' anxiety to disagree with the writer has led him into a frame of mind in which he is prepared to find that it is "manifestly improper" to charge to capital the real cost of construction, but is quite proper to charge to operation a part of the cost of construction, even though this results in carrying into the operating account items of expense incurred long before operation began or could have begun.

Mr. Riggs thinks that the writer was incorrect in objecting that "a uniform price for earthwork was used, thus ignoring the varying

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character of soil and length of haul," but he admits that there was "practically no classification in the Southern Peninsula of Michigan, or, in fact, on 90% of the mileage of the State," and his defense goes no further than to assert that "the price * * * was not much out of the way when considered as a fair average for the territory."

His criticism of the objection to the use of a uniform price list for materials, and ignoring the source of supply and the cost of delivery at the point of use, is equally forced, for it admits that "no effort was made to use different unit prices as between counties," and only contends that "in a number of cases" differences in prices were made.

The absence of an allowance for interference by labor troubles, weather conditions (which he admits are "a frequent source of annoyance, delay, and sometimes of expense"), Mr. Riggs defends on the ground that it is "an expense difficult to separate and set up," and therefore ought to be covered by an allowance for contingencies. On the same ground, he could easily carry every item of cost of replacement into the contingent account.

The two remaining objections specifically raised by the writer are squarely attacked by Mr. Riggs. As to one of them, the propriety of an allowance for carrying charges up to the time of attaining a revenue basis, has been admitted by the Railroad Commission of Wisconsin, but it is a broader question than ought here to be discussed. The writer will only suggest, at present, that in some form or other, these charges must be on the whole and in the long run met out of net operating income, and that the cheapest way, for the user of the services supplied, is to carry them into the capital account—otherwise there must be an early amortization of this item, which cannot do otherwise than to throw a heavy burden on the early schedules of charges. The language of the Wisconsin Railroad Commission on this subject merits quotation, and is as follows:*

"But new plants are seldom paying at the start. Several years are usually required before they obtain a sufficient amount of business or earnings to cover operating expenses, including depreciation and a reasonable rate of interest upon the investment. The amount by which the earnings fail to meet these requirements may thus be regarded as deficits from the operation. These deficits constitute the cost of building up the business of the plant. They are as much a part of the cost of building up the business as loss of interest during the construction of the plant is a part of the cost of its construction. They are taken into account by those who enter upon such undertakings, and if they cannot be recovered in some way, the plant fails by that much to yield reasonable returns upon the amount that has been expended upon it and its business. Such deficits may be covered either by being regarded as a part of the investment and included in the capital upon which interest is allowed, or they may be carried until they can be

* Decision and order of the Railroad Commission of Wisconsin, issued August 3d, 1909, in the case of Hill *et al.* *v.* Antigo Water Company, pp. 84-85.

Mr. Williams. written off when the earnings have so grown as to leave a surplus above a reasonable return on the investment that is large enough to permit it. When capitalized, they become a permanent charge on the consumers. When charged off from the surplus, they are gradually extinguished. (These facts alone, however, do not always furnish the best or most equitable basis for the disposal of such deficits.) Whether they should go into the capital account, or whether they should be written off, as indicated, are questions that largely depend on the circumstances in each particular case."

The other objection that is squarely opposed by Mr. Riggs is the refusal to allow for unavoidable discounts on the securities sold. Here he quotes with complete approval an unnamed writer, who contends that the impropriety of such an allowance is proven because, as between an issue of $10 000 000 in bonds (par value) at 4% and at 4½%, the 4% bonds bringing 90 and and the 4½% selling at par, there is an annual saving, in issuing the 4% of $50 000 in interest, and that, if the issue is to be for fifty years, this saving is $2 500 000, or $1 500 000 in excess of the discount. Of course, these figures are correct, but both Mr. Riggs and his unnamed authority seem strangely to have overlooked the fact that if a railway construction requires $10 000 000, it cannot be obtained by issuing $10 000 000 in par value at 90. The comparison, of course, ought to be based on the issue of enough bonds at each rate to obtain equal sums of money. As $10 000 000 in par value of bonds sold at 90 would produce $9 000 000, the following comparison is based on the issue of enough bonds at each rate payable in fifty years to secure that sum.

	Fifty-Year Bonds. 4½% sold at par.	4% sold at 90.
Amount of capital required....	$9 000 000	$9 000 000
Par value of bonds necessary...	9 000 000	10 000 000
Annual interest charge.........	405 000	400 000
If 4% bonds are used:		
Annual saving in interest..................		$5 000
Fifty years saving in interest................		250 000
Loss, original discount......................		1 000 000
Net loss.........................		$750 000

Of course, the foregoing figures are not absolutely accurate, for the real net loss in the issue of the 4% rather than the 4½% bonds at these prices would be the difference between the $5 000 annual saving in interest and the amounts which would have to be set aside annually for fifty years to produce $1 000 000, the amount of the discount, at the end of that period. But the table is sufficiently accurate to expose the curious error into which Mr. Riggs has fallen. Perhaps it will convince him that it would be better, hereafter, not to stray so far outside the field of civil engineering.

Mr. Riggs has little sympathy with those railway men who venture to express the opinion that regulation ought not to extend so far as to render it impossible to conduct the railway business in a business-like way. His animadversions on railway men in general have already been illustrated herein. He finds nothing worse with which to characterize a previous utterance of the writer's than to say of it: Mr. Williams.

"The manifest impatience with all forms of governmental interference with corporations, which so often characterizes the utterances of prominent railway officials, appears in this paper to a marked degree."

At the risk of incurring further displeasure, the writer will not omit now to observe that, in his judgment, the whole question whether railways shall be generally and officially valued, and how and by whom the task shall be performed, is primarily conditioned by the country's need of managing its legislative control of railway methods so as not to restrict unduly the flow of capital into that industry. The steady pressure for legislation during the last five years has so extended legislative regulation that, for the first time, the sturdy, frugal, conservative, "small investor" stands in the forefront of the problem. His views of the stability and future prosperity of the American railway industry now dominate the situation. What they are may be read in the facts attending recent efforts to finance necessary improvements of old and prosperous railways. It developed before the Interstate Commerce Commission during the recent hearings in connection with the proposed partial adjustment of rates to the diminished purchasing power of the money in which they are paid, that one of the greatest of Eastern railway systems, paying 8% annual dividends on its stock, which is very widely distributed, had offered new shares to its stockholders at a premium of 25%, and had found them unsalable at that figure, so that it was obliged to recall the offer and put them out at par. Other testimony disclosed the failure of one great company to obtain an offer of more than 85 for its 4% bonds, while another had been forced to go to France to raise $10 000 000, and many others have been forced to the expedient of issuing short-term notes at relatively high rates of interest. It also appeared that extensive proposals for new branch lines had been abandoned or postponed, in view of the impossibility of obtaining funds on reasonable terms.

Other testimony shows that locomotive shops and car builders are putting out not more than half of their capacity; that the supply trade is receiving no new orders. Never, since the beginnings of the American railway industry, has the American and foreign investor been so reluctant to supply necessary capital, or so doubtful of the future of railway enterprises. This fact is not due to absence of confidence in the industrial future of the American people, but is directly attributable to the unanswered inquiry as to how far the policy of

Mr. Williams. legislative control is to extend. Either this question must be answered in a manner satisfactory to the investor, or the credit of the Government must be made available for the extension and improvement of railway facilities, either through Governmental guarantees of adequate returns to capital, or through Government ownership; for adequate and properly constructed and equipped railways the public must and will have. Thus far, the American public is ready neither for Federal guarantees nor for Federal ownership; it is to be hoped that it will never be ready for either. In this situation, if a Federal valuation is to be undertaken, it is primarily important that it should be under such auspices and by such methods that the investor will not be alarmed as to its consequences. This is not a suitable occasion to attempt to lay down all the considerations applicable to such a valuation, but it ought to be perfectly clear that it must relate to value in use, not to some concept of value limited to replacement cost which excludes some of the most important elements of value (which are also those most worthy of a return, because they represent the highest and most difficult social and industrial services), in order to obtain a means of excluding these same elements from possibilities of adequate reward.

One of the most important items to be considered is the "cost of progress," which is sometimes referred to as "abandoned property," or as "obsolesence." For illustration, in the revision of the grade and line of a road, whereby the capacity of existing track is doubled, the present instructions of the Interstate Commerce Commission require the charge to operating expenses of the cost of that portion of the old line no longer continued in use. If, however, the doubling of the capacity of the line be secured by the construction of a second main track, the entire cost of the new work can be charged to capital account and paid for from the proceeds of the sale of capital securities. The latter method becomes the easier to finance, but what of the comparative results? Say, for example, the original cost of material of existing property, including equipment, stations, yards, etc., was $10 000 000, that the first main track cost $1 000 000, and that to double the capacity of the main track would require a present expenditure of $1 000 000, either for (1) a reduction of the grades and curves of the first main track, or (2) for the construction of a second main track. The increase in capacity is identical, but in the first case the cost of train service to handle the tonnage is decreased 50%, and some reduction in maintenance is secured, while in the second case no economies of operation are effected, but the expenses may be increased. Undoubtedly, Road (1) would be much more favorable than Road (2), yet the Commission says a portion of the cost of perfecting Road (1) must be charged to operating expenses, and cannot be capitalized. What general manager will dare recommend such extensive improvements when the charging of a portion of the cost to operating expenses will show the dividend as

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unearned, and thus render the securities of the company no longer legal investments for savings banks, trustees of trust funds, etc.? As an alternative, he might permit the old line to remain, and by placing thereon a few cars occasionally, could consider it as still in use, and carry it in his capital account, thus avoiding the charge to operating expenses. Thus, again, is it the method and not the result that is controlled by these instructions. What should be done is to permit the cost to be charged against the surplus accumulated during the years in which the property to be abandoned was used. This would not affect adversely the operating income of the year, and would not impair the credit of the Company.

Plainly, the instructions of the Commission tend to compel a method that is contrary to the economic law.

Obviously, any requirement as to valuation which would impose on the carrier such a result as that shown would compel the continuance of the less efficient service and prevent the progress which such replacements express. The railway business is a continuing one, and an improvement ought to be made whenever it can earn income, not only on its own cost, but on that of the property abandoned, even though it cannot afford income sufficient to wipe out the whole capital charge for the latter in a single year. There is no reason for requiring each item of capital to earn its cost in addition to its interest during its individual life. Such a requirement would cry halt to progress. It is reasonable and proper that such charges to operation should be made as far as the rapid development of the art of transportation permits, and such is the practice of every well-managed railway; but, to make the practice uniform and compulsory, permitting no exceptions and allowing no scope for individual judgment, is quite another thing. When the conditions warrant such a course, the railway ought to be permitted to adjust its accounts in a manner of which the following is typical:

	Replacing.	Not replacing.
Capital account..................	$19 750	$5 000
Additional net operating income attributable to this item......	1 000	250
Charge to operation for abandoned property	250	....
Operating gain...........	$750	$250

A valuation adjusted in recognition of this developmental need would include, in addition to the item of $15 000 for the replacement cost of the new locomotive, an item representing "cost of progress" of $4 750 for the former locomotive. It is not to be overlooked that in actual practice it would be easy to obtain this allowance by cumbering the

Mr. Williams. yards and round-houses with obsolete and superfluous equipment. The plan of Professor Adams places a premium on such a course, and there are many conditions under which it could and would be followed where it would be less obvious and more detrimental. For example, it might be that an additional track over a steep grade and a new alignment which would avoid it would cost the same. The new alignment would give greater operating efficiency, but it would require the charging off of the old line; the new track over the grade would be more costly to operate, but would leave the apparent capital unimpaired. It is such possibilities as this that are giving pause to the investors who would otherwise supply funds for the needed development of the American railway system. How far this development has so far required the abandonment of property capable of further use and having genuine capital value is indicated by available records. The aggregate capacity of all equipment has increased much faster than the increase in number of locomotives and cars. The reports of the Interstate Commerce Commission only show this information for the years 1902 to 1908, both inclusive. The average tractive power of locomotives in 1908 was 26 356 lb., as compared with 20 485 lb. in 1902, being an increase of 5 871 lb., or 28.7% per locomotive. The average capacity of freight cars in 1908 was 35 tons, as compared with 28 tons in 1902, an increase of 7 tons, or 25 per cent. Undoubtedly, the average capacity of locomotives and the average capacity of freight cars in 1908 was not less than 60% above the average capacity of 1890.

L. F. Loree, M. Am. Soc. C. E., President of The Delaware and Hudson Company, as Reporter (For United States) to the International Railway Congress, held in Paris in 1900, communicated with all roads in the United States then operating 500 miles of line, or more, relative to the capacity of cars actually in service. The result is shown in Table 12.

As a result of these improvements in roadway and equipment, the average number of tons of freight handled per freight train in 1908 was 351.80 tons, as compared with 296.47 tons in 1902, an increase of 55.33 tons, or 18.6 per cent. The average tons per freight train in 1908 was 351.80, as compared with 175.12 in 1890, an increase of 176.68 tons, or 100.8 per cent.

These improvements have not been solely or mainly for the benefit of the carriers, though there is no question that they have been prompted by railway self-interest. The new car of 40 tons capacity is but 20% longer than the old car of 13 tons, which means a great augmentation of the efficiency of the private sidings and tracks of the manufacturers, as well as the side tracks and terminals of the railway. Who would retrace the steps of progress of the last decade or of the last two decades? Yet the project to tie railway earnings to replacement cost, which makes no allowance for the costly steps in such

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TABLE 12.—CLASSIFICATION OF FREIGHT EQUIPMENT ACCORDING TO THE CAPACITY.

Year.	No. of roads reporting (see Note).	Five tons and under.	Ten tons.	Fifteen tons.	Twenty tons.	Twenty-five tons.	Thirty tons.	Thirty-five tons.	Forty tons.	Forty-five tons.	Fifty tons and over.	Total number of cars.	Total capacity, in tons.	Average capacity, in tons.
1880..	A-7	38 399	131 988	447 270	89 420	....	....	....	....	....	....	53 733	707 077	13.2
	7	38 399	131 988	447 270	89 420	....	....	....	....	....	....	53 733	707 077	13.2
1890..	A-7	16 450	71 932	182 175	651 740	441 475	548 670	....	4 000	....	50	91 231	1 916 492	21.0
	B-13	16 450	72 082	240 900	933 040	624 125	638 100	....	4 000	....	50	119 513	2 528 747	21.2
1893..	A-7	1 145	34 088	144 795	629 780	734 350	842 640	....	4 000	....	....	102 315	2 390 798	23.4
	C-13	1 145	34 238	255 795	993 840	947 500	1 112 070	....	4 000	....	....	145 440	3 348 588	23.0
1895..	A-7	355	13 978	120 435	589 140	743 975	1 011 030	70 000	4 000	....	50	104 495	2 552 963	24.4
	D-15	355	20 863	245 709	1 136 320	1 103 100	1 493 760	70 000	4 000	....	50	171 307	4 074 217	23.8
1897..	A-7	20	6 462	92 535	555 980	761 150	1 224 030	74 865	4 400	450	150	108 118	2 720 042	25.2
	E-16	20	9 407	183 189	1 029 756	1 089 300	1 822 530	183 190	4 440	450	150	174 315	4 322 432	24.8
1898..	A-7	....	1 540	94 275	523 080	721 425	1 314 840	75 320	4 480	270	50 950	108 559	2 786 180	25.7
	F-27	63 565	9 491	418 551	2 190 360	1 654 850	4 831 170	88 515	8 840	270	104 700	385 765	9 409 918	24.4

NOTE.—A—Figures cover only these roads: Reporting for 1880 and all other years, viz.:
- Allegheny Valley
- B. & M. R.
- C. of G.
- G. R. & I.
- Penn. Lines W.
- Phila. & Reading
- Wis. Cent.

B—Includes roads under "A," also:
- Ches. & Ohio
- C. G. W.
- M. K. & T.
- N. D. & C.
- Phg. & Western
- Vandalia

C—Includes roads under "A," also:
- Ches. & Ohio
- C. G. W.
- Mich. Cent.
- M. K. & T.
- N. D. & C.
- Phg. & Western

D—Includes roads under "A" and "B," also:
- Mich. Cent.
- Southern Ry.

E—Includes roads under "A" and "C," also:
- C. R. I. & P.
- Seaboard Air Line
- Southern Ry.

F—Includes roads under "A" and "B," also:
- Ann Arbor
- B. & M.
- C. R. I. & P.
- C. St. P. M. & O.
- Grand Trunk
- Lehigh Valley
- Mich. Cent.
- O. R. R. & Nav.
- Penn. R. R.
- P. B. & L. E.
- Seaboard Air Line
- So. Pacific
- Southern Ry.
- Union Pacific

Mr. Williams.

TABLE 13.—STATEMENT OF RETURN ON INVESTMENT IN ROAD, EQUIPMENT, ETC., FOR ROADS IN THE OFFICIAL CLASSIFICATION TERRITORY, FOR ELEVEN YEARS ENDED JUNE 30TH, 1909, ALSO FOR THE YEAR 1890.

Year.	Cost of road.	Cost of equipment.	General expenditures.	Material and supplies.	Total.	Operating revenue.	Operating expenses.
1909	$4 357 455 101	$686 116 206	$50 586 812	$75 550 135	$5 169 708 254	$1 032 285 890	$700 694 007
1908	4 306 902 038	669 751 320	51 324 157	86 201 748	5 114 179 263	1 049 545 984	746 575 094
1907	4 438 582 438	587 637 733	....	91 923 338	5 118 143 509	1 141 224 116	794 998 803
1906	4 269 066 800	513 028 004	....	80 479 333	4 862 574 137	1 044 552 909	714 461 452
1905	4 110 883 904	492 498 488	....	65 875 071	4 669 257 463	944 805 659	658 337 498
1904	3 906 766 459	461 941 677	....	72 240 521	4 440 948 657	899 868 519	636 217 217
1903	3 830 580 776	426 822 818	....	64 458 257	4 321 861 351	871 697 611	601 864 284
1902	3 744 205 552	389 909 755	....	50 565 290	4 184 680 597	782 975 559	528 681 892
1901	3 682 894 343	378 545 580	....	47 746 178	4 109 186 101	730 590 144	491 657 899
1900	3 620 630 187	377 156 700	....	49 940 838	4 047 727 725	698 368 829	467 462 093
1899	3 566 223 557	351 902 957	....	31 162 907	3 949 289 421	610 724 301	413 390 359
Total 11 years	$43 834 191 155	$5 335 310 738	$101 910 969	$716 143 616	$49 987 556 478	$9 806 639 521	$6 754 340 598
Average 11 years	3 984 926 469	485 028 249	9 264 634	65 103 965	4 544 323 317	891 512 684	614 030 963
1890	$2 927 221 233	$283 407 139	....	$35 262 205	$3 245 890 577	$524 767 906	$348 388 268
Total 12 years	$46 761 412 388	$5 618 717 877	$101 910 969	$751 405 821	$53 233 447 055	$10 331 407 427	$7 102 728 866
Average 12 years	3 896 784 365	468 226 489	8 492 581	62 617 152	4 436 120 588	860 950 619	591 894 072

TABLE 13.—(*Continued.*)

Year.	Net operating revenue.	Net revenue from outside operations.	Total net revenue.	Taxes.	Operating income.	Operating ratio.	Operating income. Percentage to cost of road, cost of equipment, material, and supplies.	Mileage of line owned.
1909	$331 591 883	$2 425 726	$334 017 609	$37 397 973	$296 619 636	67.88	5.738%	56 563.41
1908	302 970 890	3 446 600	306 417 490	36 021 974	270 395 516	71.13	5.287%	56 328.79
1907	346 225 313	....	346 225 313	35 876 148	310 349 165	69.66	6.063%	56 415.25
1906	330 091 457	....	330 091 457	34 863 314	295 228 143	68.40	6.071%	55 990.12
1905	286 468 161	....	286 468 161	27 675 211	258 792 950	69.68	5.542%	54 963.20
1904	263 651 302	....	263 651 302	28 091 468	235 559 834	70.70	5.304%	54 643.50
1903	269 833 327	....	269 833 327	26 537 954	243 295 373	69.04	5.630%	53 873.11
1902	254 293 667	....	254 293 667	25 297 465	228 996 202	67.52	5.472%	52 980.70
1901	238 932 245	....	238 932 245	23 797 264	215 134 981	67.30	5.235%	52 911.46
1900	230 906 736	....	230 906 736	22 616 893	208 289 843	66.94	5.146%	52 495.25
1899	197 333 942	....	197 333 942	21 692 694	175 641 248	67.69	4.447%	52 009.93
Total 11 years	$3 052 298 923	$5 872 326	$3 058 171 249	$319 868 358	$2 738 302 891	68.88	5.478%	599 174.72
Average 11 years	277 481 721	533 847	278 015 568	29 078 941	248 936 627	68.88	....	54 470.45
1890	$176 379 638	....	$176 379 638	$14 753 550	$161 626 088	66.39	4.980%	43 094.73
Total 12 years	$3 228 678 561	$5 872 326	$3 234 550 887	$334 621 908	$2 899 928 979	68.75	5.448%	642 269.45
Average 12 years	269 056 547	489 360	269 545 907	27 885 159	241 660 748	68.75	....	53 522.45

progress, is in reality a project to tie them to their present state of development and to prohibit future progress. Nor can it be forgotten that it is an inviolable law of Nature that that which does not go forward must go backward—nothing can remain stationary. Mr. Williams.

The story of the crude millionaire who wanted to know the value of the "plant" of Oxford University, in order that he might duplicate it, is not inappropriate, and ought to have some significance to those who imagine that replacement cost would tell the story of railway values. Do they imagine, because they are ignorant of them, that a great railway organization carries no traditions of loyalty, of persistence in the face of overwhelming difficulty, of generous recognition of public needs and rights, of courageous adherence to the real interests of its shareholders that inspire its personnel and provide a genuine *esprit du corps?* Do they find no superiority in one organization over another, no systematic economies of method, no especial adaptation to economic needs that has value more genuine than any replaceable element, and is at least equally worthy of compensatory return?

If there were not abundant evidence that the railway industry is not excessively profitable, there would be more reason on the side of those who continually put forward new schemes of restriction; but, not only is such evidence ample, but there is no evidence of any sort tending to establish the contrary. Limiting the inquiry to the region east of the Mississippi and north of the Ohio and Potomac Rivers, commonly known as Official Classificaton Territory, the statement in Table 13, based on the book cost of railways, with their equipment, supplies, and materials on hand, is instructive. The data are from the reports of the Interstate Commerce Commission.

The amounts shown in Table 13 as "operating income" are, as should be remembered, those earned, and not those distributed as interest on bonds and dividends on shares, which were necessarily much smaller. Bearing this in mind, it is significant that the percentage of such operating income to cost of property has not but once in the last twelve years, the most prosperous duo-decade in the Nation's history, exceeded 6%, and then only by a very small fraction; and that the average for the whole period is less than $5\frac{1}{2}$ per cent. Every one knows that the real value and the actual cost of the railway property in this region greatly exceeds its book cost, so that these percentages are undoubtedly much in excess of the real rates of net earnings to value or cost of property.

P. E. Green, Assoc. M. Am. Soc. C. E. (by letter).—It is not often that there is presented to the Society a paper which shows such thoroughness of understanding of a difficult problem, and as much real experience in its solution, as is manifested therein; and the author is certainly to be congratulated on such a logical and forcible Mr. Green.

Mr. Green. presentation of the subject. There may be some points on which engineers who have been engaged in such work cannot agree with him; but certainly it cannot be said that he has not argued very clearly and logically on nearly all the debatable questions.

Those who have not had actual experience in making a valuation of a railway company's property cannot have any idea of the enormous amount of detail and labor necessary to make such a compilation of any real value. It simply means that every detail of every structure of whatever kind must be investigated, together with the various considerations covering "intangible values," which the author has so ably discussed.

The writer was fortunate enough to be employed on the valuation of the Chicago and Northwestern Railway property in Minnesota in 1906, and possibly some details of the manner in which the actual field work of the survey was done may be of interest.

The work consisted of making a compilation from records, or from actual surveys when necessary, of about 625 miles of railway property, including several important terminals. The property had been built between 1860 and 1901, mostly in the early part of this period. The portions which had been constructed during the latter part of the period, say from 1890 to 1906, presented no difficulties, as the records were very clear and complete, but the portions constructed in the Sixties had practically no records. Some had been built by small independent companies, which were acquired later by the Northwestern System. On these old lines the records were practically nil, and those in existence were soon found to be of absolutely no use. Even on the newer lines it was found that many changes had been made within a few years after their construction, and that it was sometimes more economical, as regards time at least, to make a new survey of the property than to use the records.

After examining all the old records very thoroughly, and endeavoring to get some order and information out of them, it was decided that the only way to do the work properly was to make a complete survey and valuation of all the physical property. Several field parties were organized and also an office force, about twenty men being put on the work. The parties ran levels for profile purposes, cross-sectioned cuts where necessary, noted evidences of clearing and grubbing, of the character of the cuts, and the disposal of the material, examined the ballast for depth and character, examined the rails for age, weight, and condition, and noted the kind and condition of the fences, gates, farm crossings, planking, whistle and highway-crossing posts, culverts, bridges, and in fact every detail of construction. Advantage was also taken of the survey to re-station the lines, to paint such stations on the rails, and to set permanent posts, so that afterward the stationing could be picked up at any time with little trouble.

Mr. Green.

In this way there was accomplished much work of value to the railway company, for which there had been a demand by the division officials for years, but which had not been done because of lack of men and money.

No attempt was made to assign depreciation, as regards the rails; this was determined afterward, from the age of the steel in the track. It was necessary, however, to make quite a thorough inspection of the ties, and to note their condition, as they were replaced year by year singly as they wore out. Almost every conceivable kind of timber had been used for ties at one time or another. Treated and untreated ties lay side by side; and thus there was great difficulty in classifying them with regard to the kind of timber. With bridge ties and timbers of frame and pile bridges, there was not so much difficulty, as they were open to inspection, and had been inspected twice yearly by the Division Engineer and the Superintendent of Bridges and Buildings, and accurate records of their condition and renewals had been kept. The depth and condition of the ballast also varied very widely.

In a very short time all the men on the survey became well acquainted with the character of the work they had to do, and, as the work went on, the progress of the party day by day was very much more rapid. At the beginning of the survey, a progress of 6 or 7 miles of single track was considered a very good day's work; at the end of the survey, the parties were making from 12 to 15 miles per day.

There was considerable difficulty in setting proper values on the hundreds of buildings, large and small, owned by the railroad. Most of these buildings had never been constructed from plans, and it was difficult to calculate what they had cost originally, and what it would have cost to build them at the time of the survey. However, time books were searched, and the contents of the buildings in board feet were calculated, and, while in many cases their age was not known from any records, it was nearly always possible to find out from somebody just when they were erected.

As intimated before, the railway company derived much actual benefit from the work, outside of the accurate knowledge obtained as to the value of the property itself. Steel charts, bridge records, etc., were established, and profiles, stationing, continuous bench-levels, etc., were all re-run or re-established; thus making the engineering work of the future more consistent and uniform, and enabling more work to be done with a smaller force. New maps of all the station grounds and terminals were obtained, and all the records were put in better shape than they had ever been before.

Examination of some of the old terminals brought to light many strange and out-of-date conditions. Old wrought-iron rails of antiquated pattern, old cast-iron frogs, etc., of a pattern which had not

Mr. Green. been in general use for fifty years, were found in the track. On some of the little-used sidings, the old wrought-iron rails were so worn that the tread of the rail was entirely gone, only the web remaining to carry the traffic, and such rails were still in use.

In such a valuation, also, many items, some of considerable magnitude, were found which were extremely difficult to classify and assign to their proper place. Such a one, for instance, as a soft, sand rock deposit beside the track, which for many years had furnished engine sand. Many thousand cubic yards of this material had been excavated, but it had not gone into the roadbed as ballast, or to make fills, or to widen embankments. It would hardly have been proper to classify such excavation as grading, for it was an item of engine maintenance and train operation. This is only one of numerous problems which had to be solved.

After all the survey had been made, most of the work of compilation had to be done. Some of it had already been done in the office by the small office force, but the great mass had to be done by the men who did the work in the field. This task was of almost incomprehensible magnitude. There were thousands and thousands of items, and such a great mass of figures that the ordinary man would become lost in the maze. The data had to be checked and re-checked by men who were not accountants, and sometimes most ludicrous mistakes were discovered. However, it was at last accomplished, and the writer's recollection of the "Present Value" of the Chicago and Northwestern property in Minnesota is that it was somewhat more than \$23 000 000 for the entire mileage (about 625 miles), or an average of about \$37 000 per mile of track. Hardly any of the mileage would be called high-class or trunk-line track, but most of it might be classed as second-class or important branch-line railway.

Mr. Kuichling. E. Kuichling, M. Am. Soc. C. E. (by letter).—This paper is a very valuable addition to the literature of a comparatively new subject that is rapidly attaining great political importance, and it gives abundant evidence of deep research and thought by the author. The reasons for determining the true value of such properties, as well as the general principles of making the valuation or appraisal, have been set forth so clearly and convincingly that little can be added in this respect; hence, there is room for comment only about details.

One of the perplexing questions is the determination of the proper value of the right of way and real estate of a railroad. The land was originally acquired at a certain cost, essentially for public use, and in the course of time its value, as determined by reproduction cost, usually becomes greatly increased by the development of the adjacent land by its various owners. Without the railroad, such development and appreciation of land values would probably not have occurred, and, therefore, it has been argued by many persons that, for taxation pur-

Mr. Kuichling.

poses, the railroad lands should be appraised at only their original cost, while, for capitalization purposes, they should be appraised at a value measured by that of the adjacent land at the present time. This claim is based on the theory that the railroad is like any other piece of public work, such as a canal, highway, or pavement, which is built for the use of the public, and on which no tax is levied by State or municipality. On the other hand, it has been held by some of our Courts that a proper valuation must take into account the appreciation or depreciation of land values; but, as the opinion of a Court is not unalterable, the soundness of this doctrine cannot be regarded as permanently established.

The author states* that there can be no serious objection to this doctrine in relation to rights of way in the country and small towns, although he admits that it is subject to exceptions in the case of cities and terminal and dock properties. It will be of great interest to learn his reasons for making such exceptions in the case of the most costly lands, and whether the valuation of such lands should be more or less than that of similar adjacent lands used for other purposes. From the context the inference may be drawn that the valuation should be somewhat higher than that of adjacent similar land in the case of a steam or interurban railroad, because its holdings form a continuous strip; but to the writer this reasoning does not appear satisfactory. The statement of the Court, that "the value of land depends largely upon the use to which it is put and the character of the improvements upon it," does not necessarily involve a higher valuation of the property than its cost, and it is quite conceivable that the actual value of the property after being taken by a railroad may be much less than it was before. The only reason in such a case for maintaining the purchase price is to conserve the general valuation of the adjacent similar real estate.

In dealing with the subject of depreciation, the author has been very brief, as he did not consider it essential for the purposes of the paper. This is to be regretted, as depreciation is an important feature in every valuation, and so few trustworthy data concerning it are available. The value of the paper would be greatly enhanced if the author would give the assumed average life of the principal components of a railroad, based on some definite traffic, and normal grades and curves. Much diversity of practice in this respect prevails, and the final judgment of the numerous experts who were engaged in the Michigan valuations cannot fail to be of great interest. The same remarks are also applicable to the unit prices adopted for construction and equipment.

The subjects of expenses for organization, engineering, administration, contingencies, and non-physical values are treated very thoroughly by the author, and particularly interesting is his discussion of the

* *Proceedings*, Am. Soc. C. E., for November, 1910, p. 1507.

Mr. Kuichling. complex question of franchise value. After quoting from numerous judicial opinions, he reaches the conclusion that the franchise simply protects the owners of the property in their enjoyment of the earnings, and that its value merges into the "fair value" of the property and becomes inseparable from the other non-physical elements of value; also, that the aggregate non-physical value of the property depends only on the net income for a period of years. This method of estimation certainly has the merit of being simple, rational, and free from all hypothetical considerations. It is, however, obviously governed by the rates charged for the services rendered, and if these are likely to be altered at any time by governmental action, a corresponding alteration in the "fair value" of the property will take place.

This consideration brings us at once to the intricate question of reasonable rates, which involves the matter of reasonable design and construction of the property. In most cases the working capacity of the plant must be much greater than the average annual demand for the service performed, as so-called "peak loads" of relatively short duration must be provided for. The magnitude of these peak loads, however, varies with the subsequent development of the territory, which is necessarily conjectural; hence it follows that a comparatively large amount of capital is often invested in an enterprise for the purpose of taking care of such anticipated temporary demands, and on this investment a "fair return" should be granted. This condition is particularly noticeable in municipal water-works plants, where provision must be made for supplying water for fire service to an extent which may be several times greater than the normal hourly rate of consumption. In the case of railroads, such demands can usually be met by adding to the rolling stock at moderate expense, while in a water-works the outlay is relatively greater because the entire plant must be adapted in the outset to the anticipated maximum delivery in the course of a comparatively long period of time.

The problem of rate-making has been excluded by the author from his present paper on valuation; but, inasmuch as he is so well qualified for the task, and also because the non-physical value of the property depends mainly on the rates obtained for the service rendered, it is hoped that he will deal with this feature in a subsequent paper, thereby bringing out a discussion on the obscure subject of "fair return." It is noticeable that these phrases occur frequently in judicial opinions, but the fundamental principles on which a definite conclusion should be reached are seldom set forth clearly.

Mr. Dana. RICHARD T. DANA, M. AM. SOC. C. E. (by letter).—The solution of this problem includes practically all the factors in the general subject of economics, in which engineering occupies a large but by no means preponderant part. Mr. Riggs has done some very valuable and pioneer work in contributing this paper at this time; and the writer, in calling attention to what appears to be a radical error in it, does not wish to

be taken as attempting to detract in any way from its great value as a whole. It is most important, in the inception of such an investigation, on the part of the members of this Society, to remove from the subject the stumbling blocks as they appear. Mr. Dana.

The author makes the following statement:

"It is true that the 'value' of a property is an unstable figure, subject to fluctuations due to natural or artificial causes, and that a material change in value may occur suddenly, but the 'value' of any given property on any given date is, or should be, from an engineering standpoint, a definite sum which may not be varied or changed to suit the whim or will of the people for whom the work is done."

The fundamental conception of a value is so important in an investigation of this kind that it is worthy of careful and thorough discussion.

The appraisals which the writer has had occasion to make have generally been for one or other of the following purposes, namely:

(I) Taxation, in the interest of the community or corporation taxed;

(II) Bonding, in the interest of the banker or representative of persons who contemplated loaning money on the property;

(III) Rate-making, in order to determine what was a fair amount of money that the property should be allowed to earn for the owners.

Now, in general, a proper value for any property for any one of these purposes is different from its proper value for any of the others. This proposition is of immense significance, for the reason that, if the value for the property arrived at, on one basis, be accepted and applied for one of the other purposes, it will inevitably result in gross injury and financial loss to some one.

In attacking this problem, one must be careful to take the correct standpoint, which is not necessarily that of engineering. Engineering science is indispensable for a large part of the work, but there are other indispensables which would not ordinarily be recognized as engineering. The writer takes the view that engineering is a part of economics, rather than economics being a part of engineering.

To illustrate this point, consider two objects, one of which is concrete and simple and the other more complex.

(1) A steam shovel belonging to a railroad, costing $10 000, new;

(2) The entire railroad as an operating entity.

Assume for (1) that the shovel has been purchased recently, is in perfect condition, and that the railroad has some work for it to commence on as soon as it can be properly installed. What is its:

(I) Taxable value,

(II) Bonding value, and

(III) Rate-making value?

Mr. Dana.

(I) The tax assessor cannot properly appraise it at $10 000, because it certainly would not sell for that sum, and if the community should have to sell it for taxes the actual return minus the charges would be so much less than the $10 000 that the community's books would show a heavy loss; and this practice, if largely indulged in, would bring the community into financial straits. The community must be exceedingly conservative in its estimate, for this very reason; and, therefore, it has been customary, almost universally, to tax such articles practically on their sale value at what might be called panic prices. The company which sold the shovel to the railroad would not buy it back two days after the sale for more than the original price minus what that company considers its selling charges, say 20%; so that, in this case, even if a customer were at the door, the shovel would not be worth more than $8 000, and a fair tax appraisal could not consistently be more than $8 000 minus charges of, say, $250, or $7 750.

(II) Assuming that the railroad is a very small one, that it wants to borrow money, and desires to put up the shovel as collateral for the loan. What would be its loan value to the lender? In considering this point, it is necessary to assume that no aid is rendered by the credit of the railroad itself, but that the protection for the loan is to be furnished by the shovel only. Now, the banker will reason that, in the event of the note remaining unpaid, he will have to sell the shovel to reimburse the bank for its loan, and he will be required to consider the matter on a conservative basis. He cannot loan on the shovel up to its full value, for in the first place it is not a "negotiable security." If it were a security, with a free market on some stock exchange, he would probably loan to the amount of 80% of its value, but a steam shovel in a sand bank on a railroad is by no means as convenient of exchange, nor as easy to foreclose on as a stock certificate in a banker's box; therefore he will loan, or he ought to loan, less than 80% of its sale value, minus the selling charges. If he loans more than this, he is loaning on the credit of the owner of the shovel rather than on the shovel itself. Granted that the maker of the shovel is willing to buy it back at its full selling price less the selling cost, the maximum loan value of the shovel would be a little less than 64% of its purchase price, or $6 400. To loan more than this on the shovel would not be conservative banking.

There is another bonding or loan value to this shovel, when it is considered as part of the assets of the railroad whose bonds are to be held by the banker, under which circumstances a higher value than $6 400 would be admissible.

(III) If the value is to be determined with a view of ascertaining what is a reasonable figure that the owner of the shovel ought to be allowed to earn as a public utility organization, the problem is entirely distinct from the foregoing two cases. Assume that the railroad is

entitled to earn at least 6% on its investment in the shovel. Now, its investment is \$10 000, because that is the money that it cost; and nothing had been credited to its account, since the shovel had just been purchased and had not yet done any work. The shovel cannot be considered as being worth more than its cost, and it can easily be shown it is not worth less for rate-making purposes. Mr. Dana.

These three illustrations, which are very briefly outlined, should demonstrate the fact that there is almost no relationship between any two of the different kinds of value which are being considered.

Now, from the standpoint of the railroad as a whole:

(I) Should railroad property be taxed on the basis of what the entire railroad would bring on a foreclosure procedure? Obviously not, because the railroad is taxed in sections. The Town of Squedunk will tax the portion of the railroad that lies within that town, and will have considerable difficulty in putting down as security for its own bonds the locomotives and cars which go through once a week or twice a day at 40 miles per hour. To cover partly the flitting assets, it taxes the railroad on a franchise value. It may tax a railroad's land on the same basis that it taxes land owned by private individuals, notwithstanding the fact that when the railroad buys the right of way it generally has to pay more money per acre than the householder or the farmer. This unit cost to the railroad may be two or three times that to the farmer, yet the writer has never heard of a community attempting to tax railroad property two or three times as heavily as adjoining property used for private or commercial purposes.

(II) On the other hand, this same property is an absolutely sound asset for the railroad, and the railroad probably bought the property from the proceeds of the sale of bonds. If the public service commissions were to rule that the railroad may be allowed to issue bonds only to the amount of the taxable value of the property which is to be held as security for the bonds, the result would be an absolute paralysis of railroad construction. A bond is an obligation to pay so much interest for so many years, and to pay back the principal at the end of its term. The bondholder is interested in the absolute regularity of his interest, and in the security that lies behind the principal, and it is to-day the custom of banking houses to consider a bond well secured when, in a territory of reasonably rapid growth, the principal is earning say twice the interest on its bonds, and when the cost of reproduction is in excess of the amount of the bonds, provided that the property is in good physical condition. If it should be necessary to foreclose on the the bonds, it is then reasonable to suppose that some one else will buy it in for at least the amount of its bonded indebtedness. What can this possibly have to do with the taxable value of the track in the Town of Squedunk? One may be 1.5 times the other, or three times the other, depending on a multitude of circumstances.

Mr. Dana.

(III) The value of the property for rate-making is a complex one to determine, and, of course, there is no opportunity for a full discussion of it here. One point, however, will serve to establish thoroughly the difference between this and taxable or bonding value. If the community is prosperous and the business is a good one and honestly managed, the railroad ought to be allowed to earn a reasonable percentage, say, at least 6%, on what has been put into it. If the community should decree otherwise, then people will not build railroads for investment purposes, and all will lose money. Now, it is a well-known fact that a new railroad's earnings have to grow for several years before they are on a normal basis, and part of what the owners of the property have put into it is, for example, the interest on its cost before its earnings are on a normal basis. This may amount to a considerable percentage of the original construction cost of the property, if the business is several years in developing. Granted that the community ought to allow the property to earn a reasonable interest on what has been put into it, then the rate-making value will be very much larger than the sum of the taxable valuation of all its different parts. It will also be much greater than its bonding value, because, as a bond proposition, it can borrow money up to a limited percentage of what it is actually worth.

Mr. Hammond.

GEORGE T. HAMMOND, M. AM. SOC. C. E. (by letter).—The engineer called on to fix the valuation of public service corporation property has so little engineering literature on this special subject to guide him that he must feel grateful to the author of this excellent paper for adding so much of a kind that is very desirable.

Estimating the cost of an engineering structure in advance of its construction is one of the most ordinary professional duties, but how difficult it actually is, and how much engineers differ with one another in their estimates on the same structure! Perhaps there is no professional duty which calls for so much study and so much experience, or which tests so closely the ability and capacity of the engineer. How seldom professional estimators agree with each other; or designing engineers with contracting engineers; as witness the bids received at the public lettings of contracts when compared with the engineers' estimates of cost; and, if this is true, which no one will attempt to deny, how much more so is it probable that estimators will disagree when they attempt to place a value on works already completed, and in service, perhaps, for many years, in which various changes in value have occurred, and in which questions of fact are mixed with legal questions involving legal rights, as well as financial questions.

The tendency in all such valuations appears to be a mixing up of things in general—like the witches' stew. Everything goes into the pot and is boiled together until all becomes soup, at least until the official commission, like the witches, considers it done and ready to be served

Mr. Hammond.

up in the form of a report. It is then observed that the substance served out is of a complex nature; that the valuation of engineering structures has become mixed with other and uncertain values; that the whole value, as stated, is, after all, little better than the commission's opinion of the value; and that another commission would reach a different conclusion.

The author states that the valuation of corporation property:

"Should be the honest judgment of the men composing the commission, as to the actual cost of reproduction, present physical value, or 'fair value,' and should be ascertained by a systematic and scientific method which takes into account all the facts concerning the property, its physical value, its strategic location, its operating revenues and expenses, and its franchises, rights, competition, opposition, and all other tangible or intangible elements, which would affect values. The method of valuation should be such as to minimize or entirely eliminate all differences due to errors of personal judgment."

This, it seems, complicates actual present values with conditions which might, or might not, continue. Outside of the physical valuation of the plant, which offers the easiest problem presented, how can one fairly put a value on operating expenses and revenues, which might be affected favorably or advisedly by good or bad management, and by numerous other complex and almost incomprehensible circumstances.

The tendency of all such commissions seems to be to confuse together and mix up some things which are logically separate. Thus, the value of the plant and franchise, good will, and present investment or income value, etc., are too often taken together. The value of the plant is dependent on the cost of reproduction, and also the depreciation of the structures, as engineering structures, and should be based on present prices for which the work could be replaced, minus the depreciation, which is a question of engineering judgment and experience. The other items of value are largely dependent on the situation of the plant and its prospects as an income-producing property, and this again is a matter of opinion, in which the opinions of financiers and investors are sometimes of more moment than those of engineers. The opinion of lawyers as to the value of the franchises and the cost of the legal complications possible or probable must also enter into any seriously worthy opinion as to value.

The few salient lights in the picture of valuation, presented by the author, serve especially to reveal the darkness which involves the whole subject of valuation, estimating, and the use of cost data for such purposes, and to suggest that, with all the wonderful progress on the theoretical side of the profession, engineers have as yet advanced but little in this division of the practical side—cost data, valuation, and estimating. Engineers cannot compare the results of different estimators and appraisers without sorrow and even shame for their ignorance, or

Mr. Hammond. their incapacity to agree in the application of scientific principles and the results of practical experience to this branch of their work.

At present we would seem to be a long way from a method of valuation, "such as to minimize or entirely eliminate all differences due to errors of personal judgment."

The method described as having been used by Professor Adams seems to be at least an advance toward a logical and rational method of getting at the value of corporation property, but it must be acknowledged that we are as yet a long way from a perfect method of appraisal, even of the physical values, to say nothing of the non-physical. He held that as nothing visible or tangible gave support to the latter value, it must be determined on the basis of information secured from the income accounts of the company. This method of measure, it would seem, is not unlike the celebrated dictum on the length of the Chancellor's foot, "some Chancellors have a long foot, and some an indifferent foot, and some a short foot"; therefore, a great English Chancellor says, "the length of a Chancellor's foot should not be taken as a measure of rights in equity." Thus, if the income of the company is to be taken by the appraising engineers, or the gross income, it may have to be given a different interpretation from the net income, and if the surplus earnings depend on transient causes or on excessive rates for service, it will lead to a totally erroneous conclusion. The same may be said if the rates for service are too low, or if the company is badly managed, or is carrying a great deal of "dead wood," either in the form of property or of servants. Therefore, it seems evident that he who attempts to follow this method of appraisal must possess almost superhuman judgment of present conditions, and prescience to forecast the future, as well as a grade of wisdom and knowledge of existing conditions of trade and industry which may be also characterized as superhuman. In order to apply Professor Adams' method justly, we must know whether the company is wisely managed, whether its income is a fair income, whether its physical property is all useful and needful, whether its service is what it ought to be as to efficiency and economy, etc. We must assume an ideal condition of commerce and industry, and of property value and management, and then appraise the company's property by comparing it, consciously or unconsciously, with this ideal. Possibly this is the best method devised so far, but surely it leaves a great deal to be desired; and it is difficult to see how different engineers, on different sides of the question, representing different interests, can find any common ground of agreement in Professor Adams' method. Under such circumstances, engineers are likely to differ in their results as much as the length of the different Chancellors' feet.

Mr. Metcalf. LEONARD METCALF, M. AM. SOC. C. E. (by letter).—Mr. Riggs has done engineers, and more particularly those interested in valuation

works, a genuine service in presenting to the Society this admirable paper. Mr. Metcalf.

No shrewd observer can fail to recognize the increasingly insistent demand of the public for greater publicity in the accounting, and a larger measure of governmental control in the operation, of public service corporations. In its best form, such control will be welcomed by the corporations, as giving greater stability to investment in such property; in its worst, it may prove a serious limitation to prompt development of the best standards of service. In the water-works field, the anti-corporation movement has resulted in taking over by the public many such plants. It does not seem likely, however, that we are ready to go farther in the railroad field of operation than to demand reasonable regulation of such corporations.

While the writer has had no experience in railway management or valuation, he has devoted much time and thought to the valuation of, and determination of fair-rate schedules for, water-works properties; therefore, what he may have to say in comment on this paper may be assumed to have direct application to water-works valuation, and to railroad valuation only as the similarity in the public service rendered by these corporations may imply.

In the main, the writer subscribes heartily to the views expressed by the author and the temperate way in which he has expounded them. Space forbids discussion in detail of all the matters alluded to and so well covered by Mr. Riggs. On one important subject, however—the inclusion of the going value, or going concern value, of public service corporation property, in the intangible property values, rather than in physical plant value, and the consideration of it as an intangible value rather than as a real and substantial item of cost to the public service corporation—the writer differs from the author. It is clear, from what Mr. Riggs has said, that this is debatable ground, and, from the care and fairness with which he has expressed his views on this subject, one might almost be led to infer that he invites attack on it. It is in no carping spirit of criticism, however, that the following views are expressed.

As the writer has recently submitted to the Publication Committee of this Society a paper on the "Going Value of Water-Works," written by him in collaboration with John W. Alvord, M. Am. Soc C. E., in which a detailed discussion of this subject will be found, only enough will be said to outline clearly his point of view.

The author says:

"The physical property is that which enables the corporation to do business. Without physical property it could not produce the commodity which it sells. The amount of money actually invested in acquiring that physical property represents the measure of capital on which it is morally entitled to earn interest and profit; and, in the

Mr. Metcalf. stage of promoting and financing the enterprise, all hope of earnings is based on the amount of money required to construct the property."

He also says:

"It would seem reasonable to say that this difference between the physical value and the value based on earnings represents the 'good will,' 'established business,' or 'going value,' and all other non-physical elements of value."

In referring to going value, he says:

"* * * Yet, to fix a value on it by the method described by him [Mr. Alvord] involves going into the realm of conjecture and speculation to a degree that could never be sustained. * * * It can be readily seen that the physical present value is not always—indeed, is not often—the 'fair value.' The 'fair value' may be more, or less, than the present value of the physical property."

"* * * Is it not, then, proper to conclude that the non-physical or intangible value, composed of all these various elements of value, can only be determined absolutely by a study of the earnings and operating expenses? * * *"

He also says:

"The contention that all the different elements of non-physical value merge into one intangible value, not capable of separation, will doubtless be objected to by many engineers and corporation managers. * * *"

"The writer does not concede that 'going concern' is a proper element to consider in the physical value, as it does not represent any part of the cost chargeable to capital, and the physical valuation should be confined to the determination of capital invested."

Quotations might be multiplied. Those cited, however, will suffice to recall the author's view, and to make clear the point with which issue is taken.

Is Mr. Riggs right in his contention that going value is in fact an intangible value; that going value is not an element of real cost to the company, involving investment of capital; that going value, therefore, should not be included in physical plant value; and that the company is not morally entitled to earn interest and profit on it?

The writer contends that going value is as real an element of cost, in the property of any public service corporation, as is the cost of any portion of its physical plant. It pertains, however, to the business, rather than to the physical plant, of the corporation.

Whatever the difficulties of its computation may be, whatever the methods used—whether that adopted by the Wisconsin Public Service Commission (which is essentially one of determining the original cost of the going value and not its reproduction cost), or whether that perhaps first outlined by Mr. Benezette Williams and George H. Benzenberg, Past-President, Am. Soc. C. E., in the Middle West, and

Mr. Metcalf.

by William Wheeler, M. Am. Soc. C. E., in the East,* a method which seeks to determine the reproduction cost of the going value, rather than its original cost—the going concern value has come to be recognized, by water-works appraisers at least, as a substantial element in the cost of the plant, and hence as differing essentially from the franchise element or so-called unearned increments of value.

Is not going value in a "between" class—a middle ground between tangible and intangible values—tangible in that it has involved real cost and expenditure of money; intangible in that it is not as readily calculated as are other reproduction cost items, is dependent fundamentally on the earnings of the company, and that there is no tangible equivalent to show for the expenditure, except the existing income of the corporation? Surely its character is quite different from that of the franchise, as ordinarily found, the value of which, while real, from the rate-payers' point of view, seems to be made out of whole cloth; in short, seems to be of fictitious value.

Certainly, the conjectural and speculative character of the computations—as referred to by Mr. Riggs—involved in the determination of going value is no excuse for failure to recognize going value as a real element of cost, rather than as an intangible value. As a matter of fact, the variation in the views on going value, by engineers who have given this subject particular study, while greater than the variation in their estimates of the reproduction cost of the physical plant, is still far less than the variation in their views on franchise value.

As bearing on the proper basis for rate-making, the author's statement, that the "* * * physical property represents the measure of capital on which it [the public service corporation] is morally entitled to earn interest and profit * * *" cannot be admitted, equitably or legally; and it is not to be assumed that Mr. Riggs desired to imply that this sentence summed up his final views.

Are we not, however, approaching a basis of rate-making, predicated on the earning, by public service corporations, of operation and maintenance expenses, depreciation allowance, and return (*i. e.*, interest and profit) on reproduction cost of the property, less accrued depreciation, plus going value, plus a nominal allowance for the franchise and other intangible values of the corporation? Is it not possible that the recent depression in the business world has been due, in considerable measure, to the shrinkage in the values ascribed to franchise and other intangible value in public service corporation property?

If we are approaching such a limitation, it is the more important that the public should be educated to the fact—not theory, for it is a

* Shortly after the Kansas City Water Company case and the classic decision of Mr. Justice Brewer, and since developed by the suggestions of a number of engineers, among them John W. Alvord, M. Am. Soc. C. E., whose admirable article on "Going Value of Water-Works," presented at the Milwaukee Convention of the American Water-Works Association, held in 1909, is familiar to all students of water-works valuation.

Mr. Metcalf.

fact—that going value, or going concern value, is a real element of cost, covering an outlay in effort and money on the part of the corporation, and as such is as much entitled to earn a return (interest and profit) as is the other capital invested in plant. It is not on any items of real and necessary cost to the corporation that the public objects to paying tribute, but on the "unearned increments" and the virtual monopoly "privileges" enjoyed by the corporation and created, in large measure at least, by the public itself and by normal conditions of growth and development for which the public, rather than the corporation, was perhaps responsible—though in many cases it may be urged truly that the corporation itself, rather than the public, has been responsible for the development.

Such a basis of rating, while still dependent on sound judgment and judicial treatment, is nevertheless not beset with the speculative element involved in the capitalization theory, which, Mr. Riggs himself admits, fails as a basis of rate-making except when predicated on fair rates.

If the writer's contention, that going value is a real element of cost in the property of any public service corporation, is sound, Mr. Riggs' statement that, "It appears to be doubtful whether the Court can be construed as approving such an element of value in rate cases," and his interpretation of Judge Tayler's ruling in the Cleveland Street Railway matter,* must be challenged.

Certainly, as applied to water-works valuation, Mr. Riggs' statement is not justified. The Maine cases clearly include going value as an element of value on which rates should be predicated; by inference, so does the Kansas City case. In the Knoxville case it was in fact allowed by the Master.

In equity it cannot be doubted that going value should be included in the base on which the returns are predicated, if, as contended, it involves real cost to the company; for the company must be permitted to earn a fair return on this cost, or to liquidate it in some way, as otherwise the corporation would suffer substantial property loss—from 10 to 20%, more or less, of the reproduction cost of its property. This would be contrary to public policy, for, with such an outlook, capital would not enter this field of enterprise, except at increased rates of return, commensurate with this added hazard. To assume such increased rates of return is to provide another means of liquidating such a loss.

As to "good will," it has seemed to the writer more proper to use this term in private competitive corporation enterprises, as applied to the element of value corresponding to the going value of the quasi-municipal or public service corporation enterprises, which latter are in effect controlled monopolies. If the term is used in its more collo-

* *Proceedings*, Am. Soc. C. E., for November, 1910, p. 1523.

quial sense, such as the effect on earnings of having, in the office of the corporation, men who meet the public pleasantly, who are good "mixers," and who are active in getting business, the value is substantially included in the consideration of the income, in the manner involved by going value determination and franchise valuation. Mr. Metcalf.

The depreciation question has been discussed so fully elsewhere that the writer only calls attention to the fact that, while physical and functional depreciation only are to be considered in a review of the present physical condition of any plant, in considering a fair-rate schedule, provision should also be made for contingent depreciation, covering such items as cost incident to change in street grades or construction of subways; placing structures under ground, which were previously above ground; serious loss due to injury by electrolysis, the distribution of which over a period of years rather than inclusion in the operating cost for one year, is to be preferred, alike from the public and from the corporate point of view, from the fact that it spreads the burden to be borne by the rates, and prevents violent fluctuation in prices or valuation of the public service corporation's property. The public pays dearly for all hazards. It is wise, therefore, to pursue the conservative course in providing adequate funds to meet extraordinary conditions, and to give stability to the investment of the corporation. Moreover, such funds can be carried in a separate account which can readily be watched; any excess can be credited to future reduction in depreciation account requirements, while a prolonged deficit cannot perhaps be recovered by the corporation, in the light of the Knoxville decision.

The comment that no hard-and-fast rule can cover determination of proper depreciation allowances, is amply justified. In its final analysis, it is a matter of good judgment, experience, and judicial temper.

The author's statement that the organization, legal, engineering, administration, and general expense accounts, "should not be considered as affected by depreciation, as long as the property is a going concern," is not quite clear. Obviously, this is true with regard to all the early organization expenses, as these expenses are incurred once for all, and constitute a continuing asset similar to other elements of plant cost. If, however, the author refers therein also to the engineering and contingent item added to many of the reproduction cost items making up the physical property, exception must be made; for when an old structure, the life of which is gone, is replaced with a new structure, new engineering costs are incurred, and the engineering element of cost incident to the installation of the original structure no longer inheres in the plant. It, too, has passed away with the life of the structure, and, therefore, its cost should be liquidated, or pro-

Mr. Metcalf. vided for in the depreciation account, as well as the cost of the structure to which it was incident.

In the same way the "interest-during-construction" item is not a continuing asset, but should be liquidated with the complete depreciation of the portion of the structure to which it relates. The replacement of the structure will involve new "interest-during-construction" charges, commensurate with the time required for construction. The value of the initial "interest-during-construction" costs will have disappeared with the original structure and, therefore, should be taken care of by the depreciation account.

The method of making allowances for interest during construction, suggested by the author,* accords closely with that used by Mr. Alvord and the writer in a recent valuation of a large water-works property, in which the "interest-during-construction" charges were limited, and the contributions to depreciation account were begun, at the date on which any workable unit of the property was assumed to be available for service and to begin to earn a return on its investment cost, even though the structure, as a whole, was not assumed to be completed for a considerable period of years thereafter. Thus, for instance, it might be assumed that as soon as the supplying works in a water-works project were in operation, the investment in them and in the distribution pipe system laid up to that time, would cease to be credited further with "interest-during-construction" allowances, and would be compelled to earn interest through the water rates or income from water supplied to consumers—the fact that the interest charge could not be wholly met, immediately at this time, being taken care of in the resulting increment in going value.

Such a theory, of course, does involve a determination of the probable order and rapidity of construction of the component parts of the property, and this is usually made, in water-works valuation, in the estimate of the reproduction cost of the property.

For the sake of completeness, in reference to the legal decisions of importance in valuation proceedings, attention is called to the Pennsylvania case, Brymer *vs.* Butler Water Company (179 Pa., 231), referred to in the closing discussion on the writer's paper on "Water-Works Valuation."†

In this case Justice Williams, speaking for the Supreme Court of Pennsylvania, says:

"By what rule is the Court to determine what is reasonable and what is oppressive? Ordinarily, that is a reasonable charge or system of charges which yields a fair return upon the investment. Fixed charges and costs of maintenance and operation must first be provided for. Then the interests of the owners of the property are to be con-

* *Proceedings*, Am. Soc. C. E., for November, 1910, p. 1512.

† *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 94.

sidered. They are entitled to a rate of return, if their property will earn it, not less than the legal rate of interest; and a system of charges that yields no more income than is fairly required to maintain the plant, pay fixed charges and operating expenses, provide a suitable sinking fund for the payment of debts, and pay a fair profit to the owners of the property cannot be said to be unreasonable." Mr. Metcalf.

The Pennsylvania Court, therefore, in the words of William S. Wallace, Esq., recognizes the single standard:

"The Single Standard, according to the Brymer case, while acknowledging the full right of the public to regulate such public corporations, also recognizes as a prime factor its private character and the rights which accrue to it in that capacity, * * * and holds to what seems to me the only rational and practicable basis, that a fair return, after deducting the charges above enumerated, *is* a reasonable rate"; whereas, "the Double Standard basis of fixing a reasonable rate seems to accentuate the public side of the corporation and rather ignores the private element."

As to the propriety of the inclusion of a substantial recognition of franchise value as a basis for rating, the layman may well confess to perplexity, in the light of the conflicting nature of the two important recent United States Supreme Court opinions referred to—the Knoxville case, and the Consolidated Gas Company case—for, while substantial allowance was made for franchise value, in the Consolidated Gas Company case decision, in large measure apparently on account of its earlier recognition by the legislature, in the Knoxville case, in spite of legislative recognition of such value, and similar approval of the issue of securities predicated on such recognition, the United States Supreme Court failed to make similar allowance for franchise value.

The author's treatment of the unit price question and the contingency item is intelligent and creditable. Engineers are prone to make valuations based on "hindsight" instead of "foresight," on the assumption that no substantial difficulties in construction were encountered, when, in fact, substantial difficulties should perhaps have been anticipated, and may actually have been encountered in the original construction, record of them having been obliterated, however, with the lapse of time.

The author's definition of the value of a property, as the "estimated worth at a given time, measured in money, taking into account all the elements which add to its usefulness or desirability as a business or profit-earning proposition," suggests the advisability of recognizing the other side of the ledger by modifying his statement so as to read: "* * * all the elements which add to, 'limit, or detract from' its usefulness or desirability as a business or profit-earning proposition."

While recognizing the author's view, that there is no separate and independent method of determining franchise value, which is not

Mr. Metcalf. based on the determination of the value of the property as a whole, by capitalization methods, it must be recognized that going value may be determined independently, and may have a positive value, even though the property as a commercial whole is worth less than the sum of the physical value and the going value.

The Court, appraisers, and the author, alike recognize that there is no one method of valuation of universal application. First cost, reproduction cost, reproduction cost less depreciation, commercial value determined by capitalization, worth of the service to the consumer, and market price of the property, if such exists, all have their weight, in varying degree in different cases. Whatever may be said of, for, or against, these several methods of valuation, relates rather to their significance, and the weight which should attach to the results obtained by them, as evidence of value and of the effect of the modifying local conditions, than to the soundness of the methods themselves.

In this connection it may be of interest to refer to a recent valuation of a water-works property, in the appraisal of which the writer chanced to participate, in which there was finally placed before the board of appraisal a summing up of:

1. The original cost;
2. Reproduction cost less accrued depreciation, plus going value;
3. The worth of the service to the consumers, based on a stated assumption of reasonable increment in value in excess of actual cost, upon which a return (or interest and profit) should be earned;
4. The commercial or capitalized value, on certain assumptions based on present conditions, and also on possible future conditions which might be involved in a renewal of the City contract, which was to expire within two years.

That these determinations of value, from different points of view, had an influence in moulding the opinion of the individual appraisers, there can be little doubt; yet it is probably equally true that in no case was like weight attached to the several items or bases of valuation. Nevertheless, in the final valuation, the consensus of opinion as to the value of the property, as a whole, was remarkably close, the extreme variations in opinion being approximately 8%, more, or less, than the final appraised valuation.

Attention should also be called to the necessity, in any valuation by capitalization of income, such as that outlined by Mr. Riggs and used by Professor Adams in the U. S. Government Valuation of Railroads,* of determining whether the plant or property is in what might be termed an over-built, normally developed, or under-built condition; in short, whether the investing public has correctly gauged its momen-

* Bulletin 21, Department of Commerce and Labor, U. S. Bureau of the Census.

tary physical condition with reference to its bearing on the rates, and whether the earnings are in fact inadequate, commensurate with the service rendered, or excessive. In the long run, due weight will be given to these facts; in a brief period, they may be incorrectly gauged. In water-works properties, unfortunately, there is rarely, if ever, a market price of the securities which can be said to be credible or significant in valuation. Therefore, in the valuation of water-works properties, it is the more important that the appraisers should weigh carefully the present character of the service furnished and the momentary adequacy or inadequacy of the rates as predicated on such service, on the needs of the community, and the existing standards of the day, if full justice is to be done. **Mr. Metcalf.**

In conclusion, the writer reiterates his statement, that he has taken issue with the author in no carping spirit of criticism, but with a recognition of the difficulty and complexity of the work of appraisal, and the conviction that engineers are under a moral obligation to do an educational work in pointedly calling attention to the fact that the going, or going concern, value, of a public service corporation's property is not an intangible value representing an unearned increment, but a very real and substantial item of cost in the property as a whole. While the difficulty may be met by placing going value, as suggested by the writer, in a middle class between physical plant and intangible values, the placing of it in the same class with franchise and other intangible elements of value, as suggested by the author, may, in the judgment of the writer, do a serious injury to corporations, in failing to give expression, in such a manner as shall be clearly within the grasp of the popular mind, to the fundamental idea of the cost of developing going value. While the writer has no personal interest in the matter, on one side or the other, having served both municipality and corporation in water-works valuations, he feels, nevertheless, that engineers can do a genuine service, alike to the public and to the public service corporation, in laying stress on the fundamental elements of cost and value, and particularly those on which rates should be predicated, in public service corporation property valuation and rating.

CHARLES HANSEL, M. AM. SOC. C. E.—So much has been said on the subject of valuation of public utilities that, although the speaker has thought on the subject for ten years, and has done considerable valuation of railroad properties, he finds that he is considerably confused, for the reason that the discussions seem to cover the whole field of engineering, accounting, taxation, and economics; therefore he suggests that, in order to get down to a basis of usefulness, a special committee should be appointed to take this question under consideration. **Mr. Hansel.**

The speaker had the honor of being associated with the Michigan Commission, as a member of the Board of Review. Professor M. E.

Mr. Hansel. Cooley was selected by the State of Michigan to take charge of the work of organization, and Mr. Riggs was the engineer who organized the office and field forces. Both these gentlemen were eminently successful in that very difficult work. Mr. Cooley did this Board the honor of saying that there were so many problems coming up in actually carrying out the work (aside from the theories of taxation, rate-making, accounting, and several other things, which could be found more readily in the Auditor's office than in the Engineer's), that he had asked for the appointment of this Board of Review, to sit as a Court, and to pass on the many complex questions arising from day to day; and he had the satisfaction of coming to the Board every day and saying: "Well, now here is a condition, and how will I handle it?" Of course, actually, he knew more about it than the Board, but he was kind enough to say that he would ask for the Board's opinion. That Board adjudicated all these various questions to the best of its ability, and the speaker has the satisfaction of knowing that the valuation has stood in the Federal Courts. The subjects are so fugitive and so illusive that very much depends on the point of view.

The speaker is now engaged in the actual task of trying to place a valuation on some $300 000 000 worth of property in New Jersey, involving the most important terminals in the United States.

The valuation of public service utilities is the most profound question which has ever been before the Society, and it includes a great deal which is outside of strictly engineering questions; in fact, the discussions do not throw much light on the methods which should be followed in making valuations.

The terminology of a subject is very important; in fact, the speaker has found it so important that in his discussions with the Attorney-General of New Jersey, in reference to the Railroad Tax Law, which he has been asked to re-draft, that draft will be accompanied by a glossary, so that the meaning of certain terms used in that particular Act will be clear.

In this New Jersey work some eighty-seven engineers and assistants are employed, and for their guidance the speaker has prepared thirty-five pages of very carefully considered instructions. These instructions are accompanied by blue prints showing exactly how all field notes must be recorded, with diagrams of trusses, culverts, and the like, and all the elements of railroad construction.

The Tax Law of New Jersey states that, first, the true value of the real estate shall be ascertained; second, the true value of the tangible personal property; and the first law of 1884 stated: "and third, the value of the franchise"; but somebody discovered that there was something besides the value of the real estate, the tangible personal property, and the franchise. They did not know what it was, but there was something else; therefore, in the 1888 law they changed the third

Mr. Hansel.

division of value to read: "the remaining property, including the franchise."

As an example of one of the difficulties of determining classification, attention is directed to the term, Real Estate, which is broadly, but seldom accurately, understood.

The Interstate Commerce Commission is the highest tribunal in the land, in the matter of railroad accounting, but it affords no help in many important elements of value; for instance, under the Interstate Commerce Commission, real estate includes only such real estate (land) as is not required for railroad purposes. All land actually used for railroad purposes is classified under "Right of Way and Station Grounds."

When the engineers on the New Jersey valuation were sent into the field, it was necessary to specify exactly what elements must be described as real estate, and what as tangible personal property. The division line had to be defined accurately for the reason that all personal property is assessed permanently to the State, while, in the case of real estate, the State receives the taxes on a strip not exceeding 100 ft. in width, and the tax on all property used for railroad purposes outside this strip reverts to the taxing district wherein it is found.

The vexatious question as to whether machinery is to be considered as real estate or personal property was settled by the New Jersey Law, which says that tangible personal property shall include all machinery; but it left unsettled the question: what is machinery? After careful consideration, real estate was divided into 74 classes, and all other tangible elements were classified as personal property. Some of the items of real estate are: ash-handling machinery and the like, chimneys, cisterns, conveyors, dams, locks, lock machinery, electric wiring, piping, heating, interlocking, signaling, pavements, reservoirs, shop fittings, tanks, telegraph lines, track, track scales, transfer tables, water-works, etc., etc. Generally speaking, all items of a fixed character were included in the 74 divisions of real estate.

The difficulties of determining all the elements of real estate are mentioned simply to call attention to what at first glance seems quite simple, but on close examination is found to have great complexities.

The question of useful life depreciation, direct and indirect, due to decrepitude or obsolescence, or both, is one of the illusive questions; and then comes the value of the franchise.

The valuation of railroad property in New Jersey is further complicated by the requirements of the State Tax Law, which specifies that the value of the remaining property, including the franchise, shall be determined after the "true value" of the real estate and tangible personal property have been determined.

The speaker will not attempt a discussion of franchise values, as it is a subject which requires the most profound study.

Mr. Hansel.

The author states that he is appalled at the speaker's misconception of the method of determining non-physical value used by Professor Adams in Michigan. The speaker is perfectly familiar with that method, and, although having the greatest respect for Professor Adams' opinions, is compelled to draw attention to two important elements of that formula which are open to objection.

Professor Adams establishes his annuity on the depreciated value, rather than on the cost, or the reproduction cost, which, in the speaker's opinion, does not determine the proper annuity or reasonable fixed charges to be deducted from net income before net surplus is established. Bonds are generally sold at a considerable discount, and represent the full cost plus this discount, consequently, the interest on bonds or fixed charges will be greater than an annuity established on cost, "reproduction cost," or "present value." Would it not more nearly establish fixed charges or annuity, to take the cost plus discount and commissions as the basis on which to apply the annuity rate?

While Professor Adams' formula establishes a larger net surplus for capitalization than the method suggested by the speaker, he in effect destroys this net surplus by charging against it all betterments chargeable to income. It is quite clear that this gives the railroad company a chance to absorb all net income into betterments, and thus wipe out all net income, in which case there would be no net surplus to capitalize, consequently, no non-physical or franchise value, and the total value established under this plan would be less than if the property had not been improved by the betterments—*reductio ad absurdum.*

In reference to the question of whether or not the method of valuation should be the same, regardless of the purpose to which the value is to be applied, the speaker cannot agree with the author, and believes that it is quite consistent to establish different values for different purposes.

The completion of a large public utility, planned on such a scale as to provide for the requirements of many years to come, utilizing but part of its capacity, and earning less than its operating expenses and fixed charges, with its rates of toll fixed by law, must be considered in a different way than a well-established public utility, with business forcing it to its utmost capacity, and with tolls not fixed by law. There are many important elements bearing on this consideration of value, and the purpose of the valuation should be known before attempting to establish the value.

In New Jersey the work is complicated further by the necessity of establishing the value of 122 separate railroad corporations, and the assignment of all property outside the 100-ft. strip to each of the 450 taxing districts through which the 122 corporations, with their many branches and spurs, are operating.

Mr. Hansel.

In order to determine the quantities and materials in the permanent way and structures, nine engineer corps were organized, each corp consisting of six men. With this force the center line of the main running track was measured, and the exact distance in each taxing district recorded. Cross-sections of the roadbed were made as often as changes in the natural surface required, and accurate measurements and notes were made of all structures; and, although in many cases the engineers were able to secure the plans of the more important steel structures, the field parties were required to obtain sufficient data to compute the tonnage in case it was impossible to get these plans.

The field parties were also instructed to note the character of the land and improvements adjoining railroad property, and record such other information as was necessary for a comprehensive understanding of the conditions attendant on the construction of a railroad in that locality.

The time allotted for the completion of the work is one year, and although this is a comparatively short period in which to introduce a premium system in field work, it was decided to inaugurate such a system as would be as nearly satisfactory as possible under the conditions. A record of each field force for each day in each month was made on profile paper, using the horizontal lines to represent the number of tracks, and the vertical lines to represent distance. Two horizontal lines were allowed for single track, four for double track, and so on. One mile was allowed for each vertical division of the paper, and, in awarding the premium, there was taken into consideration, not only the extent of territory covered by each field party, but much consideration was given to the field notes, and a cash prize was awarded each month.

The results of the organization and encouragement to the field parties are shown by the very great increase in the amount of work per man of the field parties, which was nearly 300% during the time the parties were in the field.

A great many questions hinging on interstate commerce, and involving Fundamental, State, Federal, and International Law, are embraced in the broad view of the valuation of railroad properties. The movement of rolling stock through various States, and between the United States, Canada, and Mexico, and the determination of the situs and domicile of floating equipment, are subjects which, not only require considerable knowledge of railroad operation, but involve many questions not clearly determined by the Courts.

The subject is of such great importance that steps should be taken to formulate methods of procedure, and, at the Annual Meeting of the Society, the speaker will offer a resolution requesting the appointment of a Special Committee to determine the proper methods to be used in the valuation of public utilities.

Mr. Schreiber.

J. MARTIN SCHREIBER, M. AM. SOC. C. E.—Engineers and those generally interested in the valuation of public service properties are fortunate in having the valuable information embodied in this paper. Although there are some points on which the speaker differs with the author, the following remarks are only offered in order to bring out, from experience, some further phases of the subject, rather than as an attempt to criticize.

A great deal is heard about the exact cost of reproduction, also arguments in reference to the proper allowance for contingencies, probably only involving a small percentage. The speaker questions the propriety of advocating the exact cost figures. The carefully checked cost figures of reliable contractors, with first-class engineering organizations, submitting proposals on the same construction, are often found to vary from 5 to 15% from the total cost. Different organizations will sometimes be the cause of figures varying 5% or more, depending on the efficiency and experience of the corps. A clever purchasing agent will reduce an apparently precise estimate on equipment or supplies as much as 10%; on the other hand, the condition of the market may be such that the actual price paid exceeds the estimate by the same percentage. Engineers who are responsible for estimating on, and the execution of, construction projects generally add more than 10% for contingencies, as it is practically impossible to anticipate them, and a precise estimate is almost certain to fall short. It is unfortunate that it is almost impossible to sustain contingency figures on the witness stand; for that reason, probably, it would be more satisfactory to the lay mind, and to the various courts, boards, etc., which are required to pass on valuations, and do not thoroughly understand the technicalities of the situation, if engineers would drop the contingency item and modify the quantities or the unit prices.

If it is possible to estimate the exact cost of reproduction, certainly considerable variation may be expected from independent sources in computing depreciation and present values. Yet there are reputable engineers who would have one believe (assuming that they know the cost of reproduction) that by a simple field inspection they are able to compute the exact present value.

Some time ago, the speaker heard an expert testify in the interest of a certain city, for tax purposes, with reference to the value of a piece of street-railway track. He first stated the valuation for reproduction, and then the definite present value. The latter was greatly in excess of the actual value. The expert, who was an engineer of considerable standing, on cross-examination, did not know the height of rail from top of head to bottom of groove, either at the joint or any other part in its length; he did not know the exact depth of flange of the car wheels which operated over that track, the headway, or the exact weight of the cars used. He had assumed the condition of the ties, and that the

track was ballasted. Finally, he was compelled to admit that his determination of the depreciation, by simply a field inspection, was a very rough approximation. Now, it is not in every case in the past that a corporation attorney, even with engineering assistance, has been able to point out unfair testimony. Many times the speaker has heard incompetent testimony admitted, on the general principle that the witness was an engineer of note, even though his record had been made in other specialities. Too much stress cannot be put on this phase of the subject, and the speaker is glad that the author has mentioned the fact that the personnel of those doing appraisal work should be of the highest order. In the past it is probable that the failure to discriminate properly in accepting incompetent testimony (not to mention prejudiced testimony) was automatic, and this is the most important reason for much of the hostility of officials of public service properties toward all forms of investigation, as the author mentions. Mr. Schreiber.

Company officials know that they are often compelled to employ and train specialists to furnish, within fairly accurate limits, the very information which is being sought, and naturally they are skeptical about the data presented by those who, though not intimate with the property, purport to give exact cost figures. Any one who is able to point out a consistent method whereby these exact figures may be obtained surely will obtain credit for a valuable contribution toward the solution of the complex subject of valuation.

Referring to the valuation of the property of the Detroit United Railroads, mentioned in the paper, the Director of Appraisals for the city estimated that the cost of the complete appraisal of the property, which includes about 220 miles of single track, would be from $3 000 to $4 000. Approximately, $25 000 has already been spent, not including the expense sustained by the company, which furnished a large proportion of the information.

Probably correct present value estimates which include depreciation may not be even fairly approximated without intimate knowledge of the particular property, and this should embody operation, policy of management, past performance, study of historical cost (as far as the records will permit), estimated cost of construction, and actual cost of maintenance. The life of a piece of track or equipment, disregarding obsolescence and extraordinaries, generally depends on the type and details of construction, the service it has done, and the service that will be required of it. Renewals should be made when the cost of repairs reaches a certain figure, other conditions being favorable. It is a fact that able engineers, intimately acquainted with the case at issue, and employed on the same property, often have conflicting ideas in reference to the life of track or equipment, one recommending immediate renewal and another advocating longer operation.

The speaker does not intend to argue against the possibility of

Mr. Schreiber.

placing fairly accurate values on reproduction or present value, but wishes to bring out the fact that it is not as simple as the lay mind is often led to believe. Further than that, he is of the opinion that the following is essential for economical and satisfactory valuations for all concerned:

(1) There should be co-operation of the appraisers with the public service property officials, including operators, engineers, and their records.

(2) Present values should be determined by:

(*a*) Cost of reproduction,
(*b*) Mortality tables,
(*c*) Data of performance,
(*d*) Field examination.

(3) The organization for the appraisal work should be of sufficient scope, and should be allowed the time and funds which the project reasonably requires.

(4) The appraisers should be carefully selected, the personnel including men who have had wide experience in the particular class of operation; and specialists should be obtained if necessary.

Mr. Riggs states that the valuation should be the same, regardless of the principles at issue. It seems questionable to consider the fair value which involves the non-physical value in costs or tax regulations. Certainly, in the case of street railways in cities, where a percentage is levied on the gross receipts, the non-physical valuation, only representing present value, is necessary. Again, a physical present value for taxation should not include the value of paving in the street in the strip occupied by street-railway tracks. That the street-railway company often pays an arbitrary assessment tax and keeps the paving in repair, though it is in no way responsible for the wear, should be sufficient to offset any obligation for other taxes. In some States this is fixed by the Courts. The physical valuation, however, intended to be used in connection with rates, cost, or capital regulation, should include the cost of paving the railway strip. Referring further to the question of including the paving in the physical valuation of street railways, in the case of a decision of R. W. Tayler, Arbitrator, in the proceedings between the Cleveland Electric Railway Company and the City of Cleveland, on a basis of a renewal of franchises, Judge Tayler said:

"Paving represents actual money expended. It belongs to capital account, and in its depreciated form is worth all the allowance that I have given it."

Also for rate-making and the capital regulation some consideration is certainly due to obsolescence and change of art, while in taxation they should not be included.

In conclusion, the speaker is optimistic enough to believe that the problem of physical valuations will be solved satisfactorily for all concerned. Co-operation of officials of the public service properties, reliable testimony, with a better understanding by the Courts, will certainly tend to clarify the situation. Non-physical values are very difficult to determine, and their intelligent treatment will require some well-defined procedure. Mr. Riggs' valuable paper will go a great way toward producing a correct idea of the general proportion, and will, no doubt, assist in the formulation of proper methods for valuation. Mr. Schreiber.

Clinton S. Burns, M. Am. Soc. C. E. (by letter).—The author is to be congratulated on the detailed care shown in the presentation of this subject. Perhaps few engineers who have not been called on to cope with the subject of valuation of properties, realize or appreciate the real complexity of the many varied problems encountered in work of this class. To those who are engaged directly in appraisement work, this paper will be a welcome contribution to the literature on the subject. Mr. Burns.

The author's statement that if a commission of engineers is directed to report the true cost of reproduction, depreciation, or present value of a certain property, the final figures should not differ, whether the report is to be used as a basis for reorganization, sale, rate purposes, or taxation, is open to argument. It seems proper that, if a property is appraised in order to fix a selling price to a Government or municipality exercising its right to purchase, the final figures should be based on current prices of labor and material, because this does no injustice to either party. It is evident that if the seller secures payment for his property based on current prices, he may, if he desires, reinvest the proceeds of the sale in similar enterprises at current prices, so that thereby he secures the same benefits, whether prices are high or low.

It is equally evident that if the purchaser (the municipality) chooses to purchase the property, the right to purchase must be exercised at the particular time permitted by the franchise. If prices chance to be abnormally high at that time, the municipality is exactly on a par with what it would be if compelled to build its own plant at that particular time; while, if prices be abnormally low, the same relative situation still exists. There seems, therefore, to be no possible injustice to either party in using current prices, when the object is a sale or transfer of the property. However, in determining a proper value as a basis of rates, another factor must be considered. It is inexpedient and against public policy to make frequent changes in the rate charged for such commodities as water, gas, or electric current. Theoretically, the rate could be fixed each year, based on an annual valuation of the property, thus permitting a high rate one year and perhaps an abnormally low rate another year; but, practically, this

Mr. Burns. is impossible, for, aside from the inconvenience of such a cumbersome system, no community is well enough informed as individuals to comprehend any reason whatever for ever raising rates. Raising rates is invariably accompanied by a wave of indignation. However, it is apparent that a series of rates based on an annual current price valuation of the property would average exactly the same, during a term of years, as though the property were valued once for all on the basis of the average prices of labor and material for the same term of years, and the rate based on the one valuation thus determined.

If the object of the valuation is to afford data for taxation, the same argument applies as in a case of fixing rates. It thus seems proper that the object of the appraisement should be taken into consideration before it is determined whether to use average prices, or current prices, of material and labor; and, if this is correct logic, the final figures must differ according to the object in view; but, having determined the proper unit prices to be used throughout any appraisement as being the most equitable for the object in view, then, as the author well says, the appraiser must not allow personal prejudices or fancied conditions to influence his course. Above all, an appraiser must not be afraid of his client. He must not allow his personal judgment to be swerved by the latter's desires. It perhaps seldom if ever occurs that an appraiser, representing a municipality, or State, is subjected to this unconscious influence, inasmuch as his employer is merely a temporary public official, and consequently he has no client to fear. He goes into the work with a full knowledge that his employer knows little or nothing of the subject, and his only desire is to reach results which will be unquestionably fair to both parties.

On the other hand, the appraiser who is chosen by the owner of a plant takes hold of the work with a feeling that he is expected to report a value as favorable as possible to his client, and this feeling is reflected in the report, regardless of how sincerely or conscientiously he tries to avoid it.

One of the most intricate and yet interesting problems in appraisement work is the computation of the "going value," or "business value" which should be allowed in addition to the physical value.

In considering a competitive enterprise, such as a railway serving a community in competition with another independent railway, this problem must be treated in a different way than in a non-competitive business, such as a water-works, gasworks, electric plant, street railway, or similar enterprise operating under the protection of an exclusive franchise, or under natural conditions equivalent to an exclusive privilege.

In considering competitive enterprises, it is manifest that a railway operating under conditions more advantageous than its competitor possesses an intangible value equal to the measure of that advantage.

It is not clear, however, whether it is more proper to say that the railway possessing the advantage has a positive going value, or whether the less fortunate one has a negative going value. Using the rule formulated by the author, being that of Professor Adams, with some modifications, it is evident that many properties would show negative going values; but, as pointed out by the author, the Courts hold that public service corporations are entitled to earn: Mr. Burns.

(*a*) Operating expenses;
(*b*) Expenses of maintenance and running repair;
(*c*) **Taxes;**
(*d*) A sinking fund to cover depreciation and obsolescence;
(*e*) A reasonable profit on the fair value of the property.

It is improbable that a reasonable profit on the fair value of the property could be construed to mean less than the interest or revenue from a like amount of Government bonds or other non-taxable securities.

This ruling of the Courts fixes the rates at such a figure as to preclude the possibility of a deficit; from which it must follow that a negative going value cannot be created by a compulsory reduction in rates, for such action would be confiscation of property to the extent of the negative intangible value thus created; that is to say, if the Courts are right in the above ruling, then all intangible or going values are positive, and must be determined by using the most unfavorably situated railway as the basis of computation in determining the question of reasonableness of rates; and the rates in turn must be reasonable and proper before they can be applied to determine the intangible value. This raises an interesting and far-reaching query. Assume that a negative going value is the result of real competition between two roads such that the "fair value" of the less fortunate competitor is 20% less than its physical value.

If rates are based on this valuation, are they really fair rates? For, suppose the rates had always been maintained at a point where the less fortunate road could just support its physical valuation. Clearly, no rate could then be enforced which would compel it to operate for less than a reasonable profit on the fair value of its property, and the fair value under this assumption is 25% greater than before, due to no effort of its own, but simply to the fact that its competitor has not cut rates, and has thereby preserved the original "fair value" of the less fortunate road, and at the same time increased its own positive going value by an equal amount.

In view of this analysis it is doubtful if it is ever proper to consider the existence of negative intangible values, although it is true that the commercial value does fluctuate, and may be less than the physical value, due to rates which are too low, perhaps, or due to other temporary causes.

Mr. Burns. The method quoted from Mr. Alvord for determining going value applies to non-competitive enterprises only, as was stated by Mr. Alvord in his paper before the American Water-Works Association. This method is open to the criticism that the forecast of the business of the older works, and of the new hypothetical works as well, is reduced to a monetary value, based on the present rates, regardless of whether or not such rates are reasonable. Rates are subject to legislative control in many States, and there is absolutely no assurance that any other State may not adopt legislation at any time permitting regulatory ordinances to be enforced. Therefore, any forecast of the value of future business must be based on reasonable rates, for otherwise it is merely an unwarranted estimate based on a fond hope.

Taking into consideration the fact that rates must be reasonable, either by virtue of present laws or laws which may become effective at any time, perhaps in the immediate future, going value may well be defined as the present worth of the amount by which the anticipated profits of a going plant, operating at reasonable rates, exceed the present worth of the anticipated profits of a similar hypothetical starting plant, operating at those same rates. With this conception of going value, it is impossible for a non-competitive property to have a negative going value, and every operating plant has a positive going value, even though operating at a loss.

The whole problem hinges on the question of "what is the reasonable rate or proper return," and this should be determined in the aggregate as the starting point. The Courts have persistently dodged the issue, and properly so, whenever that question has arisen, leaving it for consideration in each particular case, depending on the stability of the business, the hazard involved, and various other local factors.

It may safely be conceded that this fair profit is something in excess of the return from Government bonds, and for the purpose of this discussion it matters not what figure is assumed as the fair profit—whether 5, 6, or 10%, or what-not—the theory is the same in any case. This is perhaps best explained by a practical illustration:

Take, for example, a water-works system, the physical present value of which has been determined by the method of reproduction to be \$1 000 000, and denote the going value by the unknown quantity, x; suppose, further, that 6% is considered a reasonable return on the "fair value"—not yet determined, the "fair value" being \$1 000 000 plus the going value, x. Therefore, the rates must be such as to produce in the aggregate an amount equal to the operating expenses, maintenance, taxes, sinking fund, and depreciation, and still have a profit of 6% on the fair value of the property. The anticipated profits of the going plant, therefore, are exactly 6% of $(\$1\,000\,000 + x) = \$60\,000 + \frac{6\,x}{100}$ per annum. The anticipated profits of the hypothetical

starting plant will be negative at the start, and gradually increase, finally reaching a maximum of $\$60\,000 + \frac{6\,x}{100}$ per annum. Mr. Burns.

It must be remembered that, in estimating the operating expense and income of the starting plant, as well as the going plant, the figures must be confined rigidly to the plant as it is found at the date of valuation, and in no case should any account be taken of income or operating expenses due to probable future extensions of the distribution system. Many appraisers overlook this point, and predicate the anticipated profits of the going plant on the past growth of the income account, forgetting that a considerable portion of this growth is due to extensions into new territory, and not to any material increase in revenue from the territory already served. To include income from new territory in the forecast of income is just as fatal an error as to include the anticipated expenditure of new capital in the present physical valuation. Either of these procedures is really an estimate or appraisement of some other plant, rather than the one actually under consideration.

To complete the numerical illustration, suppose it is determined that the time required to construct the hypothetical starting plant is 3 years; that a portion of the plant is put into operation at the end of the second year, taking over fire-hydrant rental equivalent to \$20 000; that the revenue from private sources aggregates \$20 000 during the last year of construction; that the expenses of operation, maintenance, taxes, and depreciation amount to \$30 000 during this year. After the time of completion of the plant has elapsed, it has the total credit for fire-hydrant rental, and it is assumed that the revenue from private sources and the cost of operation, maintenance, taxes, and depreciation increase as shown in Table 14, which illustrates the method of computing the going value, and gives the resulting value for the case just stated.

Therefore, $171\,005 + 0.2597\,x = x$; hence, $x = \$231\,000$. This result is based on the assumption that the starting plant earned no interest during the construction period. If an allowance for lost interest during construction has been made and added to the capital account already being included in the physical appraisement of \$1 000 000, then this must be charged back against the going value found above. This is clearly evident, because the calculations to determine going value date from the beginning of the construction period, and the lost interest during construction, therefore, is provided for in the result. Most appraisers allow an item for lost interest amounting to the legal rate of interest running for half the construction period, which, in the illustration under discussion, would be \$90 000; deducting this sum, if previously included, gives \$141 000 as the going value.

Mr. Burns.

TABLE 14.—COMPUTATION OF GOING CONCERN VALUE, BASED ON REASONABLE RATES.

Year, dating from beginning of construction.		Legitimate profits of the going plant.	Hydrant rental taken over by starting plant.	Domestic revenue of starting plant.	Interest on the starting plant.	Operation, maintenance, taxes, and depreciation on starting plant.	Total difference in anticipated profits of the two plants.	Present worth factor.	Present worth of the excess of anticipated profits of the going plant.
Construction period.	1st	$60 000 + 0.06x$	0	....	If the physical valuation contains an item for lost interest during construction, the same amount must be credited to the starting plant as interest earned.	0	$60 000 + 0.06x$	95.2	$57 120 + 0.0571x$
	2d	60 000 + 0.06x	0	....		0	60 000 + 0.06x	90.7	54 420 + 0.0544x
	3d	60 000 + 0.06x	$20 000	$20 000		$30 000	50 000 + 0.06x	86.4	43 200 + 0.0518x
Business development period.	4th	60 000 + 0.06x	40 000	55 000		50 000	15 000 + 0.06x	82.3	12 345 + 0.0494x
	5th	60 000 + 0.06x	40 000	80 000		65 000	5 000 + 0.06x	78.4	3 920 + 0.0470x
	6th	60 000 + 0.06x	40 000	90 000 + 0.06x		70 000	0	74.7	... .

Total going value = $171\,005 + 0.2597x$

$x = \$231\,000$

There seems to be no good reason for allowing lost interest during construction as an item in the physical valuation of a property, any more than for allowing all of the lost interest, up to the time when the property begins to yield a return equal to the rate of interest. It is one of the problems in finance, and is much better treated as an element in the going value, as shown in the above illustration. **Mr. Burns.**

One of the most difficult factors on which to agree in computations of this nature is the element of time required for the hypothetical starting plant to acquire the business. Were it not for this uncertainty, going value could be computed with mathematical precision by the method suggested.

In determining the physical valuation on the basis of cost of reproduction, such items as cost of taking up and replacing street paving over the pipe lines, cost incurred by reason of sewers and drains encountered, interference due to electric wires and conduits, interference of traffic, and other metropolitan conditions which add greatly to the cost of construction, must be allowed. Wherever such metropolitan conditions exist, there must also be present a corresponding necessity for the use of water under pressure. People use water because of necessity or convenience, and not on account of any feeling of obligation or loyalty to the water company.

If highly developed metropolitan conditions are present, new business will be acquired in the hypothetical starting plant much more rapidly than where such conditions are yet to be developed. For this reason the problem cannot be based on the early growth of the same plant, and, there being no exact duplicate of conditions in existence elsewhere, the estimate of time required for the business development period is purely speculative, and must be assumed with great care and judgment, else injustice may be done to one party or the other in the resulting going value.

It is interesting to note that, in the Michigan appraisal, the allowance of a percentage for contingencies was bitterly contested by the railroads as improper. Probably every appraiser who has been connected with rate cases has seen this same item strenuously insisted on by the corporations.

The author's query: should a corporation which is compelled to abandon appliances while yet serviceable, in response to public clamor, be allowed any item of value in the appraisal on account of such appliances, seems to be best answered in the negative. If the appraisal is for the basis of making rates, the corporation is fully compensated by the fact that its depreciation account provides for all abandoned machinery, and the average past depreciation is usually considered a fair criterion of the future. If the appraisal is for purposes of taxation, it would seem improper to levy tax on abandoned or rejected machinery or equipment. If the appraisal is to determine the present

Mr. Burns. value of a property for sale under condemnation proceedings, it is likewise difficult to conceive any reason for allowing any present value on account of property abandoned or rejected, and, indeed, if such abandoned material had any value at the time of its removal, it is more than likely that such value was converted into cash at that time.

The statement that no appraiser would be justified in placing a going concern value on a property 3 years old, or 10 years old, unless the net earnings were such as to indicate that the property had a commercial value in excess of the physical property, is questionable. "Commercial value" is not exactly synonymous with "going concern value," for, as usually considered, the term "going concern value" represents the difference between a dead structure and a live one. A property might be compelled to operate temporarily at rates insufficient to return the legal rate of interest on the physical value of the property, and while this condition continued, its commercial value would be less than its physical value, and yet this same property is worth more while running than if operation ceased and the business was allowed to die.

Mr. Gillette. HALBERT P. GILLETTE, M. AM. SOC. C. E. (by letter).—In common with others who have written on the subject of appraisals, the author omits consideration of one of the most important elements of the cost of producing the property of a public service corporation, namely, the development expense.

Development expense is the deficit in "fair return" on the investment during the early years of operation, while the business is being developed to a point that will yield a "fair return" on the investment. Unless this development expense is charged to the capital account as fast as it occurs each year, it should draw compound interest up to the end of the development period. Development expense might be regarded as a part of the non-physical value of a plant, and a few years ago the writer so regarded it. Latterly, however, he has come to see that it does not differ one iota in principle from "interest during construction," and, therefore, is properly a part of the cost of production or of reproduction of the property. During the construction period, interest on the investment is charged, and properly so, as a part of the physical cost. Does this interest cease the day after operation begins? Not a whit. The owners of the property are entitled to a fair interest—a "fair return"—on their money, from the day it is invested. At first they receive it in the form of "interest during construction," which is charged to capital account. After operation begins they must either be allowed to earn more than a "fair return" during the fat years following the development period, or the deficit below a fair return incurred during the development period must be treated exactly like "interest during construction" and added to the capital account. If public service corporation managers have chosen the first of these two methods, it does not relieve the appraiser of the duty of

Mr. Gillette.

adopting the second method; for the object of appraisals for rate-making purposes is to limit capital to a "fair return" on the investment. In brief, if there are to be no "fat years," then every "lean year" must be credited with its deficit as fast as it occurs.

This, the writer concedes, is a radical departure from such precedent as already exists, but we must not overlook the fact that we of to-day are establishing the precedents for appraisals in the future. The whole matter of valuations for rate-making purposes is still in a nebulous form, as far as the public, and indeed, as far as the Courts, are concerned. In the end it will devolve upon engineers to establish logical methods of appraisal. To do so, they must be able to look on the problem both as engineers and as jurists. Up to the present, however, this broadness of vision has not characterized most engineering appraisers, nor is it to be wondered at when the Courts themselves are in a maze.

A great deal has been heard lately about "going concern value." Ultimately, the Courts will hold that, as far as rate-making is concerned, there is no such thing as "going concern value" in the present meaning of the term. "Going concern value," in the final analysis, consists of two elements: First, development expense (as previously defined), and, second, capitalized surplus earnings. Surplus earnings are ascertained by deducting from net earnings both taxes and a low rate of interest on the investment, equivalent to interest on bonds. Many factors may affect surplus earnings; but, that "going concern value" consists largely of capitalized surplus earnings, cannot be denied. What are surplus earnings? The public replies that they are mainly the result of extortionate charges. This is doubtless correct in many cases; hence, any investigation of costs which has for its object rate-making must inevitably lead to repudiation of that part of "going concern value" which is based on surplus earnings, if the surplus is at all large. In a word, we reason in a circle if we capitalize surplus earnings, calling the result "going concern value," and then undertake to use "going concern value" as one of the factors in judging the fairness of rates. To express the problem mathematically, we cannot solve for a variable when the variable is allowed to exist on both sides of the equation. Yet that is precisely what some rate-making bodies are trying to do, and it is precisely what the Courts have often attempted to do.

To escape this confusion there is but one possible step, and that is to eliminate "going concern value" entirely. We must first determine the element of cost which the writer terms development expense, and we must regard this item as a part of the cost of reproduction. We must next cease to consider small rates of interest as being a "fair return" on this cost of reproduction. When first-class mortgages draw 5%, it is folly to talk of 6% as being a "fair return" on capital invested

Mr. Gillette. in a business enterprise, especially when this 6% is figured on the actual cost of reproduction of the property. It may be that 7% is an ample "fair return" in some cases, but in others 10% will be found none too much, considering the small size of the business and the risks involved.

The writer will not at this time discuss methods of determining how a "fair return" should be estimated, but, in general, the process should be as follows: From the gross earnings deduct the operating expenses and taxes to obtain the net earnings. From the net earnings deduct a small rate of interest (equivalent to interest on bonds) on the cost of reproduction. The remainder is profit, and should be expressed as a percentage of the gross earnings. This percentage of profit can then be compared with similar percentages made by merchants, manufacturers, farmers, and other capitalists, and then it can be determined logically by comparison whether or not the profit made by a public service corporation is "fair." We must adopt this method of attacking the problem or we shall inevitably drive capital away from railway and other fields of public enterprise.

The writer estimates roughly that a profit of 10% on gross earnings, as above deduced, is about the same as a direct return of 7% on the cost of reproducing the average steam railway.

In a recent appraisal of a street-railway system, the writer determined the actual development expense of the property, deducing it from the accounting records. It was an astonishingly high sum, even assuming only 7% on the cost of reproduction as being a "fair return." During his appraisal of all the railways in the State of Washington, for the Railroad Commission, the writer made a similar study of development expense, but this was not included in his estimate of the cost of reproduction, as it was then regarded as being a part of the "going concern value" and he was not commissioned to ascertain the "going concern" value of the railways. Not a single railway, as far as he knows, has ever presented to a State Railway Commission, or to the Interstate Commerce Commission, an estimate of its development expense along the lines indicated. Instead, the railway companies have talked in general terms of long construction periods—often claiming 20 years or more—and of great expense incurred in building up the business, and of franchise value, and of a score or more of nonprovable costs. The consequence is that they have frequently lost entirely the one great item that they are clearly entitled to, namely development expense, which is an item which can be absolutely proved from their accounting records, and, therefore, rests not on the "hot air" testimony of experts, but on facts that are incontrovertible. In like manner, other public service corporations have often signally failed to prove the full worth of their properties, because their claims for "going concern value" have been ignored entirely. When a franchise expires, the "going concern value" is usually looked on by the public as worthless, nor is this view to be wondered at.

Mr. Gillette.

Mr. Riggs proposes adding to the physical value a minus "going concern value," and he is logical in doing so, if it is conceded that values for rate-making rest on profits; but this the writer does not concede for an instant. Values for rate-making cannot rest on the very thing that it is aimed to regulate, to wit, the rates charged. Until engineers and public service commissions and Courts free themselves from this confusion of cause and effect, there can be no rational theory of rate-making.

Values for rate-making must rest primarily either on the actual costs of the production of a property or on estimated costs of reproduction, including therein both interest charges during construction and the sequel thereto—development expense.

Of almost as great moment as the item of development expense is the question of depreciation. The author, in common with most engineers, holds that depreciation should be deducted. This is a consequence of regarding a public service plant as if it were a machine bought in a second-hand store. A public service plant is a device which is intended to perform a given service forever. It is true that its parts are subject to wear, and must be renewed from time to time; but the plant as a whole is everlasting, or practically so. Managers of public service corporations, perceiving this fundamental truth, have rarely established sinking funds for the redemption of any considerable part of the plant. In a great railway system the renewal of a freight car is not a proportionately larger item of expense than is the renewal of a tooth in a steam shovel bucket owned by a contractor. This fact, coupled with the permanence of the railway plant as a whole, has led railway owners to make no provision for a return of the money lost in depreciation. Railway ties in a large railway system inevitably reach a condition such that their average age is exactly half the life of the average tie. Shall a sinking fund be provided for ties? If not, where does logic place a line of demarcation? When does an element of the railway plant attain a condition of sufficient importance to warrant "writing off" some of its value from the capital account? The facts are that railway managers have not "written off" anything worthy of mention for depreciation, and, in the writer's opinion, they have been perfectly logical. Consequently, the operating expenses have been much less than they would have been during the early years, had a sum been placed annually in a sinking fund. Therefore, the development expense, as deduced from the accounting records, is less than it would be if a sinking fund were provided; and the amount of this difference is precisely the amount of the depreciation. In other words, if depreciation is to be deducted from the cost of reproduction, it must be added to the development expense ascertained from the accounting records; so that, in the final analysis, depreciation should be ignored entirely in any appraisal of a public service corporation where the object is either rate-making or purchase of the corporation by the public. One

Mr. Gillette. qualification to this statement is needed, however, and that is that the depreciation shall not have gone far enough to result in an average age of plant less than half the life of the plant—that being the ultimate normal operating condition.

Engineers have a duty to perform, in making an appraisal of the sort under consideration, which is judicial in its character and should not savor in the least of the pawnshop. The engineer engaged by a public service commission should not for an instant make it his object to "beat down the price," no matter by what far-fetched theory he may effect the result. Nor are engineers inclined to do this, except when they regard themselves merely as agents of the public by whom they are employed. Unfortunately, many appraisers have as yet failed to realize that there is a vital distinction between the dealings that should exist in public affairs and those that actually exist in private matters involving the purchase and sale of property. In the latter case, the buyer usually takes every possible advantage of the helplessness of the seller. Is the seller ignorant? See that he remains so. Is the seller hard-pushed for money? Grind down the price accordingly. Does the seller offer goods which are a bit shop-worn? Dwell on that fact, to the exclusion of all else. Such are the tradesman's arts, and such, the writer fears, have been the arts of some appraisers of public service property.

The writer believes that, under one form of agreement or another, nearly every kind of public service can be more economically and better performed by a public service corporation than by the public itself through employees directly hired. But if America is not to pass speedily into Government ownership and operation of all public utilities, there must be a pronounced change of attitude on the part of the public toward capital now invested in public service corporations. Even as engineers, we are apt to be unconsciously influenced in our attitude toward public service corporations, not only because of the present public attitude, but because we are often put to great inconvenience by the ill-considered resistance of the corporations whose property we are called on to appraise for the public. Our duty plainly consists, first, in regarding a public service corporation as a public agent, and, second, in allotting such values that this public agent will receive a full and fair return for every dollar judiciously and honestly spent in building and developing its property. In carrying out this plan, the writer finds it wise to study the entire financial history of a corporation, going carefully through both the construction accounts and the operating accounts from the beginning.

The desirability of analyzing the actual costs of construction, betterment, and operation of public service corporations, preparatory to estimating the cost of reproduction, cannot be too strongly urged upon appraisers. Unfortunately, many corporations refuse access to their

records, or claim that the records are too incomplete to be of value. However, when they realize that from those very records can be deduced one of the largest items of cost of reproduction, namely, the item of development expense, they are certain to show as much willingness as they now show aversion to disclosing their records. Mr. Gillette.

The writer has recently completed an appraisal of a street railway system, the managers of which placed at his disposal the entire accounting and engineering records. From these the development expense was deduced, and forms an item which can be demonstrated in Court, if need be, instead of being the subject of unsupported "expert testimony." As far as the writer knows, this is the first time that a street railway corporation has voluntarily opened all its books for use in an appraisal which may be made public. May it not be one of the harbingers of a far-sighted action on the part of public service corporations, which will result eventually in eliminating entirely the hostile attitude of the public toward its accredited agents?

Reverting again, and finally, to the question of development expense, it will be seen, after study, that the method of deducing it from the accounting records provides for every possible item. The cost of advertising, the cost of colonization, and canvassing by agents engaged in building up the business tributary to the corporation, the cost of developing an efficient business organization and an efficient plant—every possible item of developing the business finds accurate record in the development expense deduced from the accounting records as outlined. This may not be apparent at first glance, but a little consideration proves it to be so. If, for example, $20 000 has been spent annually for ten years in advertising to secure business, the operating expenses have been increased exactly $20 000 for each of the ten years. Consequently, the annual deficit below a "fair return" on the investment has been made $20 000 greater each year than it would have been had no expense for advertising been incurred. In other words, the deficit below a "fair return," which is the development expense, shows automatically the amount spent for every such item as advertising. The writer regards this automatic register of development expenses as being one of the most important features of his method for determining such expense. It removes the entire problem from the realm of guesswork and expert testimony, and makes it a problem in engineering economics. It involves no question as to whether or not the existing rates charged for freight, or for any other service, are fair.

Arthur L. Adams, M. Am. Soc. C. E. (by letter).—This paper, in spirit, diction, and contents, is a masterly presentation of the best thought and argument, by engineer specialists and the higher Courts, concerning this difficult subject—a presentation which only one intimately associated with the question for years, as has been the author, could hope to make. It is of special interest, too, because it deals Mr. Adams.

Mr. Adams. fundamentally with the Michigan railroad valuation, now ten years old, and deservedly considered somewhat ancient in the evolution of what may be termed the logic of valuation methods. The frank acknowledgment of the now apparent deficiencies or errors of that work, notably in the defective method and resulting under-valuation of real estate, as well as the upholding of that which still appears to the author to be sound in principle, are excellent manifestations of the constructive and judicial spirit so necessary to the making of any substantial contribution to the art.

Unanimity of opinion in matters of detail, even among those specializing in this line of practice, cannot be expected, especially in a general discussion. Details must receive their emphasis from local coloring and local conditions. Making allowance for these local conditions in Michigan and other contiguous States—notably conditions of population and flat topography—and remembering that the basis of the paper is a railroad valuation for purposes of taxation, and not a water-works appraisal for annual rate-fixing in a semi-arid region of rapid development, or some other widely differing utility, it seems to the writer that the author has been singularly fortunate in giving expression to views with which specialists will for the most part agree.

The limitations of the logical application of the methods suggested, however, are not sufficiently defined. Early in the paper an effort is made to avoid the necessity for this, and to simplify the treatment by limiting the scope of the paper in the following language:

"This paper is confined to a discussion of the methods which should be used in arriving at a correct figure of cost of reproduction and depreciation—it does not take up questions involving the propriety of those figures when reached. The propriety or legality of using such figures as a basis for an assessed valuation, as a basis for rate-making, * * * will be conceded no place in this paper."

Such a restriction, however, seems to the writer to leave the subject much confused. It is impossible to judge of the propriety or soundness of a method of valuation while ignoring its purpose and failing to point out the limitations of its logical application. To confine discussion to a consideration only of cost of duplication and depreciation of physical properties is presumably an attempt to avoid the difficulties incident to the application of such results to specific purposes, and is in line with the frequent argument of some attorneys in litigated valuations, that the engineer must not encroach on the province of the Court by having, much less expressing, any idea relative to the application of his figures to the final solution.

With this doctrine the writer has no sympathy. The engineer is essentially an economist, and no one is more fully qualified to aid, either directly or as an adviser to the Court, in the final determination of value for specific purposes, provided he is trained in the construc-

Mr. Adams.

tion, operation, and valuation of such properties as are under consideration. To accept any less responsibility than this is to become party to inferior measures leading to popular misconception, and is justified only as a practicable first step toward the final realization and acceptance of the larger duty.

All suggested methods of valuation should be subjected to close logical analysis, with a view to their purpose. The unsuitability of the method used in the Michigan appraisal to many classes of appraisals is apparent, and can be readily indicated. Much space is given to justifying the appraisal of all so-called non-physical elements by the capitalization of the residue of net earnings after allowing interest on the investment in the physical properties. This the author refers to as Professor Adams' method. The addition of the physical to the non-physical values, as thus determined, is supposed to give the value of the property as a whole. It is evident that it gives, by indirection, the same total valuation as would be obtained by the direct capitalization of net earnings without any determination of physical values, *per se,* and, as a method, is therefore not what it purports to be. Since value, by this method, is in reality dependent on earnings, it follows that where rates are fixed by governmental authority, with the property value as the base, as is done annually in California in cases of privately owned water and lighting plants, the method suggested is without logical application, and the property values of such corporations must be determined and justified on other or modified grounds. Hence the necessity for dealing with such elements as so-called "going concern," franchise, and other possible assets, each independently, as is usually done in water-works appraisals, instead of collectively, as in the Michigan appraisal.

It should be made clear, therefore, that the method used in this railroad appraisal, for the determination of non-physical values, simply reduces the whole to one of capitalization of net earnings, and presupposes no governmental regulation of rates with the value of the property as the base; and, unmodified, has a comparatively narrow range of application.

The author seems to see difficulty ahead in dealing with rate-making by this method, for he says, near the close of his paper: "There are many intricate problems in connection with a valuation for rate-making or taxation which really belong to these undertakings, not to valuation," but, in stating some of these difficulties, he does not point out the impropriety of determining value by capitalizing that (earnings) which it may be the object of the valuation to determine and fix.

Regulation of rates by governmental authority, which means their limitation to that which is reasonable and just, will probably in the future be the purpose in the making of most valuations of the property

Mr. Adams. of public service corporations, and no methods or rules for the making of appraisals can be considered as being at all complete or fairly comprehensive which do not meet the logic of such an end.

If capitalization of net earnings is to determine railroad values for rate-fixing, whatever the process, it must presuppose a fair and equitable rate, thus following the rate, instead of the rate following the property value. This is but a shifting of the difficulty; for, what constitutes a fair and just rate, irrespective of the value of the property used, is at least as difficult of determination as is the property value, irrespective of its earnings. Valuations, to be useful, must have their purposes carefully predetermined, that the right application of principles may be made.

Perhaps nowhere more than in California has thought been directed along this line, for the organic law of the State for thirty years has required the annual fixing of rates for water and light companies by public official bodies, and many important cases involving rates and valuations of large properties, chiefly in later years, have been tried. Unfortunately, the most important and best tried of these have not yet reached the United States Supreme Court. The result, thus far, is too long a story to be told now, but it may be said that capitalization of earnings in any form is not regarded as a logical basis of value under such conditions. Franchises, as they exist here, are not regarded as having value, unless from unusual circumstances. "Going concern" value is recognized, but its money measure is sought through other channels than present net earnings.

The author's emphasis on the necessity for eliminating the personal equation, as far as possible, is commendable, but a large exercise of discretionary judgment is inseparable from the process of appraisal. The fullest investigation of all pertinent facts should be made. Too much must not be expected from rules and formulas. They are education only. Governing principles must be understood, and subsequent procedure the writer cannot better express than in the words substantially as used on a former occasion:* Having considered the various factors likely to influence the value of any property under consideration, and having summarized the results, it will remain to determine the varying degrees of importance and weight to attach to each, and to decide, in view of all the attendant circumstances, what the amount is on which the company is entitled to receive a suitable return. This final solution can never be reduced to a mathematical formula applicable to all cases. The inquiry will have established approximate limitations, both as to maximum and minimum, but there will then usually be found remaining quite a wide intervening field for the exercise of discretionary judgment.

* "The Principles Governing the Valuation for Rate-Fixing Purposes of Water-Works Under Private Ownership." By Arthur L. Adams. *Journal*, Assoc. of Eng. Societies, Vol. XXXVI, No. 2.

Mr. Adams.

That the final result will depend to some extent on the personal equation, does not of necessity detract from its worth. It only shows the greatness of the problem, which requires for its solution the exercise of faculties higher than the application of mere formulas and mere routine, faculties which are rooted in laborious thought, in ripe experience, in moral worth.

A word concerning the use of experts on work of this class: Most valuations grow out of or grow into cases at law. Under the prevailing order, the litigants secure the services of the necessary expert appraisers, who, in the course of examination, are subjected to processes usually much better calculated to magnify than to harmonize differences, and to cloud rather than to clarify issues, to the detriment of the record, the confusion of the Court, and the attempted discredit of the witnesses and their profession. Self-defense is calculated to lead witnesses into undue reliance on rules and mathematical formulas, as direct means of obtaining the desired result, instead of aids for the final exercise of a right judgment as to the real value of the property for the purpose intended, simply because it is easier in dealing with attorneys to justify mere mathematical processes than to support opinion resting on considerations of a general character, not always readily measureable in figures. This tendency leads also to under-valuations. A change in the process of court procedure relative to such expert evidence is needed, and the influence of the Profession, both individually and collectively, might be used to secure the appointment of such witnesses at the instance of the Court, instead of the litigants, to the great advantage, both of society and of those more immediately concerned.

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PAPERS AND DISCUSSIONS

THE WATER-WORKS AND SEWERAGE OF MONTERREY, N. L., MEXICO.

Discussion.*

By Messrs. James D. Schuyler, David T. Pitkethly, and George S. Binckley.

Mr. Schuyler.

James D. Schuyler, M. Am. Soc. C. E. (by letter).—For completeness of detail and wide range of subjects of general interest to engineers, this paper is certainly one of the notable contributions to recent engineering literature. It is a minute and painstaking record of the successful accomplishment of construction work under unusual climatic conditions and difficult circumstances, and reflects credit on the author, not only in his capacity as an engineer, but as a faithful recorder of facts. It was particularly fortunate that he was an eyewitness of the disastrous and extraordinary flood which swept through Monterrey, destroying many lives and much property, and has thus been able to give an intelligent estimate of the maximum discharge of the river during the height of the flood wave of August 27th-28th, 1909, when the rate of run-off per unit of area of water-shed drained reached an amount which has seldom been equalled or exceeded, as far as reliable records extend. It is worthy of note that works deriving their water supply from the source of such torrential floods should have survived with so little actual damage, and with scarcely any interruption of service. The repair of all damages to the system was estimated to have cost not more than $20 000.

As Mr. Conway did not assume charge of construction until May, 1907, he was spared the responsibility of deciding on the general plan

* This discussion (of the paper by George Robert Graham Conway, M. Am. Soc. C. E., published in *Proceedings* for December, 1910, and presented at the meeting of February 1st, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

of securing an abundant supply of pure water from sources permitting of delivery by gravity under adequate pressure for fire protection—a responsibility which devolved on the writer, assisted by G. S. Binckley, M. Am. Soc. C. E., Mr. Conway's predecessor, as Chief Engineer. Not only the water-works, but the system of sewerage and sewage disposal by broad irrigation were subsequently carried out on the plans submitted to the State Government by the writer in 1906, and given provisional acquiescence at that time. Mr. Schuyler.

There was no lack of water at hand for the supply of a city of that size, as there are large perennial springs which flow out of the travertine of the plain, and are used for irrigation in the valley below the city. One of the largest of these, near the civic center, has a normal flow of nearly 30 cu. ft. per sec.; another nearby, also within the city limits, flows some 10 or 12 sec-ft., while both the Estancia and Robalar springs, but a few miles below (shown on Plate CLXXX), discharge more than 20 sec-ft., as nearly as memory serves. Besides this supply, the water to be developed by sinking shafts in certain parts of the plain, as demonstrated at the brewery and elsewhere, was apparently a reliable source of large volume.

To utilize these sources, however, would have involved condemnation of the water-rights in the case of the springs, depriving present owners of the use of the water, and this Governor Reyes wished to avoid. Besides, it would have necessitated pumping the water for the city in perpetuity, an expense which the Governor was equally anxious to save; hence a gravity supply was made the prime requisite of the plans.

Until the concession was granted, and for a year or more afterward, it was assumed that an adequate supply could only be obtained by the storage of the flood-water of the Santa Catarina River in a large reservoir; and the earlier plans of the concessionaires were based on the construction of a high masonry storage dam at the upper end of the "narrows," where the river turns from a western direction to a course almost due east, between high vertical cliffs of limestone. The concession distinctly provided for such a dam, and among the plans on file in the State Capitol is one prepared by the late E. Sherman Gould, M. Am. Soc. C. E., for a masonry weir across the gorge. Samuel M. Gray, M. Am. Soc. C. E., also filed a plan and report proposing a capacious, shallow, storage reservoir near the city, to be filled by a large flood-water canal from the Santa Catarina Cañon.

Although the writer could not have anticipated the occurrence of floods of the magnitude of the one of August, 1909, which would surely have destroyed any reservoir built in the Cañon, he was unable to endorse the storage plan of water development, chiefly because of the uncertainty of the water-tightness of the reservoir in a cavernous limestone formation, and also because of the probable impurity of water

Mr. Schuyler.

draining from such extensive goat pastures. He, therefore, urged the development of the underflow of the river, which was manifesting itself in the springs referred to. Mr. Binckley secured two Keystone drilling machines and proceeded to profile the bed-rock at Santa Catarina Cañon and at San Geronimo, the two places on the stream where the river flows between walls of rock *in situ*. At both sites the strata were standing nearly vertical across the channel, and, by careful sampling and testing, it was found that in both locations there were thick strata of limestone so highly silicious as to be insoluble, and hence free from caverns. From this determination it was concluded that all the water which appeared in the valley below must pass through the sections where the borings were made. The results of this drilling, however, proved conclusively that the depth to bed-rock at either place was too great to permit of a masonry dam being considered as practical, and demonstrated the inadequacy of methods which had been used in the earlier investigations when dams were regarded as feasible.

The results have also shown that the subterranean supply at the lower cross-section of the river, at San Geronimo, is abundant, and can probably be increased to an indefinite degree by continuing the filtration gallery; while at Santa Catarina the same type of development can be made for a high-source supply, although requiring a long and expensive tunnel and conduit.

Mr. Pitkethly.

DAVID T. PITKETHLY, ASSOC. M. AM. SOC. C. E. (by letter).—Having been engaged on the design of sewerage systems for some years, the writer finds this paper of peculiar interest, particularly the sewerage portion. There are some points in the design, however, which do not appear to be clear.

The system is described as "strictly separate," and yet the sewers are designed to run half-full, providing a capacity of 200%, the 100% basis, or 380 liters per capita, being 90%, or 180 liters, in excess of the calculated water supply of 200 liters per capita.

It has been the writer's practice to design sanitary sewer systems on the basis of the water consumption, and to assume the whole daily amount to reach the sewer in 16 hours, thus providing capacity sufficient to care for the maximum or wash-day flow without causing the sewers to run above the calculated hydraulic gradient, which should be placed within the pipe so as to provide air space for ventilation under all circumstances.

The practice of calculating sanitary sewers to run half-full is a good one when ground-water is expected in sufficient amount to fill the remaining portion of the sewer, but when no ground-water, or roof-, or surface-water is allowed to enter the system, or all precautions are taken to exclude such, then the system may be designed so that the expected maximum, or wash-day flow, will fill the sewer to the desired hydraulic gradient.

The method of ventilating the sewers does not seem practicable. The houses are principally of one story, and yet the stand-pipes on the sewers have openings 25 ft. 9 in. above the sidewalk. Are the ventilating or vent pipes of the house plumbing carried to a height to balance this, or will these chimneys draw the air from the house drains and fresh-air pipes, breaking the seal in the so-called disconnecting traps, thus causing the circulation of air in the house piping to be downward through the sewers instead of upward through the fresh-air inlets and vents, as designed? Mr. Pitkethly.

It is interesting to note that crude sewage, as well as the liquefying (septic) tank effluent, is to be applied to land for irrigation purposes, but the application of crude sewage without any attempt at removing the suspended matter, or the effluent from the septic tanks where only a partial removal occurs, seems to be bad practice.

The author states that:

"The degree of purification in the tanks was relatively unimportant; the object to be obtained consisted chiefly in distributing on the land an effluent which would be innocuous and clear."

How he expects to obtain such an effluent by passage through screens, detritus tanks, and septic tanks only, is more than the writer can understand.

The removal of suspended matter in a septic tank depends on the strength of the sewage, the time of retention, the time elapsing between cleaning, the presence of trade wastes, etc., and seldom exceeds 38 per cent.

The subject of septic tanks and their effect on sewage is discussed in the "Fifth Report of the Royal Commission on Sewage Disposal" (England, 1908), and the following extracts, relative to the application of crude sewage to land and the effect of septic tanks on sewage, seem apropos:

"23. * * * There are also many cases in which crude sewage has been passed over land, but the evidence shows that land treatment of crude sewage is liable to give rise to nuisance by the accumulation of solids on the surface of the land. Moreover, in some cases these solids are apt to form an impervious layer, which interferes with the aeration of the soil, and so impairs the efficiency of the treatment."

"31. * * * At that time it was claimed that the septic tank possessed the following, among other, advantages:

"That it solved the sludge difficulty, inasmuch as practically all the organic solid matter was digested in the tank.

"That it destroyed any pathogenic organisms which there might be in the sewage."

"32. As regards the first of these claims, it is now clearly established that, in practice, all the organic solids are not digested by septic tanks, and that the actual amount of digestion varies to some extent with the character of the sewage, the size of the tanks relative to the volume treated, and the frequency of cleansing.

Mr. Pitkethly.

"At Huddersfield, Mr. Campbell estimated that about 38 per cent. of the solids were converted into gas or digested; * * * while at Birmingham, Messrs. Watson and O'Shaughnessy say that the figures available indicated a digestion of not more than 10 per cent. of the suspended matter entering the tanks."

"33. As regards the second claim, we find as a result of a very large number of observations that the sewage issuing from the septic tanks is, bacteriologically, almost as impure as the sewage entering the tanks."

Messrs. Winslow and Phelps, in their interesting paper, "Investigations on the Purification of Boston Sewage,"* quote a suggestion made by Stoddart (1905):

"He finds, in a septic tank of several compartments, a considerable deposit of sludge in the first compartment, giving a fairly clear supernatant liquid, which in the last chamber of all undergoes a secondary decomposition, leading to the throwing down of an additional precipitate of offensive sludge."

What took place in the case referred to by Stoddart corresponds to the author's observations of the liquid leaving the tanks in a clarified condition, but the secondary decomposition must take place in some manner, and, when it does, a nuisance seems to be unavoidable where no provision is made to care for it.

In view of the experience of others, some further treatment seems to be necessary. Such treatment should include disinfection, as no method of disposal yet devised has succeeded in reducing materially the pathogenic germs usually to be found in sewage and tank effluents.

If the crops to be irrigated are to be eaten, uncooked, by mankind, then disinfection at least is imperative.

Mr. Binckley.

George S. Binckley, M. Am. Soc. C. E. (by letter).—Mr. Conway's admirable paper is of special interest to the writer, as the entire general design of the system, as well as the extensive hydrological studies and final selection of the sources of water supply, was completed during 1906 through the joint labors of the writer, as Chief Engineer, and James D. Schuyler, M. Am. Soc. C. E., as Consulting Engineer.

In this work, the writer had the rare privilege of dealing from its inception with the problem of designing a complete and somewhat extensive system of municipal water supply and drainage, unhampered by any existing works to which the new systems would have to be adapted. It would probably be difficult to find in the United States a city of 85 000 inhabitants, previously totally lacking either a water supply or sewerage system, which, under a consistent and harmonious design, has been provided with both in the degree of completeness and structural excellence exemplified in the works at Monterrey.

The few important changes or amplifications made in the original design, and the manner in which its detail has been executed is

* Water Supply and Irrigation Paper No. 185, p. 125.

naturally most interesting to the writer, and this excellent paper should be of very substantial value, particularly to engineers engaged on similar work in Mexico or Spanish America. Mr. Binckley.

The very novel construction method adopted by Mr. Conway in the roofing of the South or Guadalupe Reservoir, seems to the writer rather to invite criticism, and the fact that in the subsequent construction of the roof over the rectangular Obispado Reservoir the customary monolithic concrete construction was apparently reverted to after experience with the separate-unit plan previously used, would indicate that Mr. Conway reached the same conclusion.

The original design of the circular Guadalupe Reservoir contemplated just about the same arrangement of columns and roof support as that actually used, but the writer had expected that the columns would be cast in place, and that the system of primary and secondary beams would be filled at the same time as, and integral with, the roof slab, the reinforcement being placed in accordance with what may be described as conventional practice. The writer believes that the efficiency of the concrete and steel placed in this manner would be notably higher than under the system actually adopted, which, in effect, is pretty much the same as constructing the supporting system of units of cut stone. If, with all the elements of structural weakness involved in the multiplicity of mortised joints, discontinuous reinforcement, etc., this construction is strong enough, it would seem that an important reduction in the dimensions of the members could have been effected by monolithic construction and continuous reinforcement, without sacrifice of strength.

The comparison, in Table 7, of the costs of these two reservoirs, is interesting, but very moderately illuminating, as the comparative unit cost of the most important element in their construction—the concrete—is not given. The total excavation cost for each reservoir is practically the same, and the general expense, engineering, and cost of fittings and accessories presumably so, but the total cost of the Guadalupe Reservoir as given is \$19 000 (pesos) in excess of that of the Obispado Reservoir, while, in the latter, there were 756 cu. m. more concrete. This certainly indicates a much higher cost of concrete per unit as laid in the South (Guadalupe) Reservoir. An actual comparison of the cost per unit of concrete laid under the two systems would be instructive.

The writer is interested to observe that the same system of subdrainage used by him in the construction of the reservoir for the provisional supply of water from San Geronimo, has been used by the author in the Obispado Reservoir. This arrangement of drains under the floor of the reservoir at San Geronimo was devised as a safeguard against damage to the lining through the accumulation of water inside the impervious bank against its back.

Mr. Binckley.

It was realized that, in such a climate as that of Monterrey, perfect water-tightness of the lining might be difficult to secure or maintain, and, if leaks existed, a sudden draft on the contents of the reservoir might result in serious damage through the static pressure exerted against the lining of the sides or upward thrust against the floor. In the writer's opinion, such a system of drains is an important element, as not alone the fact but the quantity of leakage may be determined, and danger of saturation of the supporting bank avoided—a matter of importance where, as is sometimes the case, the material of such a bank is unfit to resist the effects of saturation. The author does not state whether or not this safeguard was omitted in the Guadalupe Reservoir. Incidentally, however, the matter of saturation of the bank is not important in either reservoir, as the material of which these banks are constructed is such that settlement or failure through saturation is out of the question. It may be remarked, however, that in fixing the angle of the sides of the Guadalupe Reservoir at 60°, the writer contemplated the same system of constructing the bank as he used in that of the San Geronimo Reservoir. In this case, the bank was built up by spreading the material in thin layers, wetting down, and rolling and puddling by the passage of the ox-carts used for the transportation of the material, the wheels of the carts, and especially the cloven hoofs of the animals, producing a most excellent effect. The inside slope was built up in this fashion to a much lower angle, and with a top width considerably in excess of the finished dimensions. The excess material was then picked off to the line, and exactly to the slope. Thus the finished slope presented a surface which was compacted to a degree impossible to attain at or near the surface of the bank as built, and presenting a support of the best possible character for the concrete lining and coping.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES CYRUS KING, M. Am. Soc. C. E.*

DIED JANUARY 13TH, 1911.

Charles Cyrus King was born at Bellmont, Franklin County, N. Y., on October 2d, 1845. He entered Cornell University in 1871 and was graduated in the course of Architecture in 1875. Like other self-reliant and resourceful men of his day, he worked his way through the University, and, in spite of this handicap, was graduated with high attainments as an Architect. Among Cornellians he will be most familiarly remembered as a member of Cornell's first 'Varsity crew in the Rowing Association of American Colleges at Springfield, Mass., in 1873, and also in the regattas of 1874 and 1875, at Saratoga.

Mr. King's first work after graduation was with the Sailors Snug Harbor, of Staten Island, and the noteworthy buildings of classic architecture which were erected during his connection with that institution stand as lasting monuments to his memory and attainments.

In 1886 he became one of the original incorporators of the C. W. Hunt Company, of Staten Island, and since that time he has been a Director and also Chief Engineer of the Company. Although an Architect by profession, his profound and deliberate mind made him an Engineer of unusual ability, whose judgment was highly valued by his Company, and he was in the midst of his usefulness and mental vigor when he died.

Mr. King was modest and retiring, to a fault, but his close friends and those who had the privilege of meeting him at home will always remember his charm of character. He designed and built his artistic home, and enjoyed it for eighteen years with his devoted wife who survives him.

His funeral, held at the family residence, West New Brighton, Staten Island, at 2.30 P. M., on Sunday, January 15th, 1911, was attended by old comrades and business associates who took this last opportunity of expressing as tenderly as possible their love and admiration for a departed friend.

Mr. King was elected a Member of the American Society of Civil Engineers on September 2d, 1891. He was also a Member of the American Society of Mechanical Engineers.

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* Memoir prepared by John N. Ostrom, M. Am. Soc. C. E.

SAMUEL McMATH ROWE, M. Am. Soc. C. E.*

DIED MAY 22D, 1910.

Samuel McMath Rowe was born in Dearborn County, Indiana, on February 8th, 1831. He went to La Salle County, Illinois, about 1842, and lived at the place where the Town of Sheridan is now located. Prior to the Civil War he was a Surveyor in and about the same locality.

In 1868 he located the line of what was then the Ottawa, Oswego, and Fox River Valley Railroad, and in 1869 and 1870 was a member of the firm of Rowe and Jackson which was engaged in the work of constructing that line.

In 1872 Mr. Rowe became associated with Dr. J. R. Zearing in the firm of Rowe and Zearing, which was engaged in the construction of the Texas and Pacific Railway, from Dallas, Tex., eastward about 75 miles to Longview.

In 1874 this firm was awarded the contract for the construction of a railroad bridge across Trinity River, for the Texas and Pacific Railway Company, at Dallas, Tex., and also to extend the track 10 miles west thereof. In the summer of 1874 the firm moved to Sherman, Tex., and commenced to build the Trans-Continental Line of the Texas and Pacific Railway, laying it to a point about 80 miles east of Sherman. Moving to Texarkana, they continued the line westward, connecting with the part they had already built.

In 1875, Messrs. Rowe and Zearing were employed on the construction of the Orleans and Pacific Railroad.

Somewhat later, or about 1878, Mr. Rowe was engaged by the Santa Fé Road, in connection with track-laying, etc., near Florence, Kans. In the latter part of 1878, he was retained as Engineer in Charge of Water Service on the New Mexico Extension of the Santa Fé Road from La Junta south, in which position he remained until about 1880 or 1881.

In 1882 Mr. Rowe was appointed Resident Engineer of that railroad, with headquarters at Las Vegas, N. Mex., and while acting in this capacity in 1885 was put in charge of the construction and operation of the timber preserving plant at that place. This was the first important plant of this kind in the United States.

Early in 1887, he was appointed Chief Engineer of the Atlantic and Pacific Railroad, and while holding this position, the "Red Rock" Cantilever Bridge across the Colorado River at The Needles, California, was built under his supervision.

* Memoir prepared by J. A. L. Waddell and A. A. Robinson, Members, Am. Soc. C. E.

In 1891 Mr. Rowe moved to Chicago and organized the firm of Rowe and Rowe, Consulting Engineers, the firm consisting of himself and his son, the late Robert D. Rowe, M. Am. Soc. C. E.

From 1891 until the time of his death, Mr. Rowe was engaged in work connected with the preservation of timber; and his "Handbook of Timber Preservation" is the result of some twenty-five years of careful and exhaustive study and practice. He labored to perfect the methods and appliances of the process, studying many principles connected with the operation of timber preserving. His business was to design and install timber preserving plants, making the plans and specifications, supervising the construction, and inspecting the work. Twenty-seven plants have been built, for which he either furnished plans and specifications or had supervision of the installation, and in most cases both. His work in this line for the most part was done for the various railroad companies of the United States and Mexico. He was also employed by the United States Government in the capacity of Expert in the Forestry Service, devoting a part of his time to the work during 1905, 1906, and 1907.

He was associated with J. A. L. Waddell, M. Am. Soc. C. E., during the construction of the South Halsted Street Lift Bridge over the South Branch of the Chicago River, which was completed about 1894, and in the preliminary work of the Northwestern Elevated and the Union Loop Elevated Railroads, of Chicago.

Mr. Rowe made a careful study of the subway question, and did considerable work in connection with the subway and harbor enterprises in Chicago. Later, he became a Director and Officer of the Chicago Subway Arcade and Traction Company, and was connected with and active in this company up to the time of his death; in fact, he attended a meeting of the company only a day or two before his final illness, which lasted about ten days.

Until the time of his death, Mr. Rowe was an exceedingly active man, and took part in many business affairs. He was an actual witness of the wonderful growth of the City of Chicago from a town of a few thousand inhabitants to a metropolis of about two and one-half million people; and he always took an active interest in, and had a thorough understanding of, the civic problems which confronted that city.

Mr. Rowe had many warm friends in the Engineering Profession, and also outside of it, who appreciated him for his sterling honesty and genial kindliness. He was a good citizen, a devoted husband and father, and a thorough and painstaking engineer.

He died of pneumonia on May 22d, 1910, at the advanced age of seventy-nine.

Mr. Rowe was elected a Member of the American Society of Civil Engineers on September 2d, 1885.

ARTHUR KEDDIE MACFARLANE, Jun. Am. Soc. C. E.*

Died November 1st, 1910.

Arthur Keddie Macfarlane was born at Beckwith, Cal., on July 19th, 1887. His parents were Donald and Kate J. Leggett Macfarlane. He was named for Arthur W. Keddie, the first Chief Engineer of the Western Pacific, who is sometimes spoken of as the "Father of the Beckwith Pass and the North Fork Transcontinental Railroad Route." It seems to have been a coincidence that the youth thus named should have shown an interest in the Engineering Profession. His early life was spent at Beckwith, and a later period at Oroville, where he was graduated from the Oroville Union High School in 1905. In 1909 he received the degree of Bachelor of Science from the College of Civil Engineering, University of California.

Like many Western students, he spent some time in practical work before and during his college period. As early as 1903 he was with the location survey party of the Western Pacific. In succeeding years, for three different summer seasons, he held various positions in surveying parties working along the Feather River Cañon and across the old Salt Lake Desert.

On his graduation from the University, in May, 1909, Mr. Macfarlane accepted a position with the Benicia Iron Works, at Benicia, Cal. His ability was promptly recognized by this company, which sincerely mourns his loss and has given no uncertain testimony as to his faithful and efficient service. In his unfortunately short career he distinctly showed ability to work with other men to gain their respect and confidence, whether they were his superiors or his subordinates. His employers state that he was admired and respected by everybody connected with the Benicia Iron Works. This quality certainly would have helped him to speedy success.

For the Benicia Iron Works he designed a series of saw-tooth roof mill buildings of steel frame with concrete covering on metal lath; later, he had responsible charge of their construction, also of the installation of machinery. At the time of his death he had about completed plans for a system of handling work quickly and efficiently in the shops and yards. He felt his responsibilities; was always on hand, and anxious to see things go ahead. At times he had difficulty in getting efficient labor. This influenced him to compete with his men to spur them on and interest them in the construction. It was this zeal, in one sense, that may account for his accidental death, for it was during the performance of his duties as Superintendent of the works that the accident occurred. A guy wire of a hoist had become entangled,

* Memoir prepared by C. Derleth, Jr., M. Am. Soc. C. E.

and all his men being otherwise employed, Mr. Macfarlane attempted to free the wire himself. It loosened with a jerk sufficient to throw it upon a live wire of the Bay Cities Power Line which ran near a wall of the building. A current of 1 200 volts passed through his body.

The young engineer can afford to learn lessons from this man. In the field and shop it is always necessary to exercise care. On the other hand, he must be prompt and ready to meet danger, for no engineer can be successful who is equipped with theory and office room experience alone. He must learn to understand his workmen. He must appreciate the qualities of the materials he uses in construction. He must understand processes of manufacture and of machine installation. Such things he can study best in the field, in the trench, and in the workshop.

At the University of California Mr. Macfarlane made an enviable reputation as a student, but particularly as a man. He was an officer of the Student Civil Engineering Association, and in his sophomore year was President of his class. He received other marks of distinction from his fellow students. These indicate the temper of the man. He was high-minded, straightforward, and honorable. He took a keen interest in his University, and remembered it after he left his Alma Mater. Unlike some engineering students, he was a student of literature and broadly read. For several terms he was President of the Calimedico Club, one of the student organizations which at California represent the Eastern Dormitory Institutions.

Mr. Macfarlane was elected a Junior of the American Society of Civil Engineers on October 5th, 1909.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXVII. MARCH, 1911. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE PRESENT-DAY PUMPING ENGINE FOR WATER-WORKS.

By Charles Arthur Hague, M. Am. Soc. C. E.

To be Presented May 17th, 1911.

In the development of the modern water-works pumping engine, the refinement of tools, reduction in cost of manufacture, better and more appropriate designing, more efficient and economical shop management, have gradually led to higher and higher steam economy with lessened first cost. The commercial element has interfered more or less with the professional; nevertheless the general results show a marked improvement during the past twenty years. With the larger plants, more of a specialty in designing to suit the conditions would have produced better results. In the older and wealthier communities there is a growing conviction that low first cost is not the paramount consideration, while low cost of maintenance and high economy of fuel are. The commercial pumping engine for water-works purposes has been brought to a high level, and has been divided into four distinct types, each to suit conditions imposed, including pressure, quantity, location, first cost, fuel, maintenance, etc. These types, and rather close approximations of cost under average conditions, are as follows:

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Cost of Pumping Engines Complete, with Foundations, Piping and Appurtenances, per Million Gallons per 24 Hours Capacity.

(1) Compound-condensing, low-duty engines, horizontal	$2 300
(2) Low-duty triple, condensing, horizontal	2 800
(3) Cross-compound, condensing, horizontal	3 300
(4) High-duty triple, condensing, vertical	4 800

The first and second are non-rotative or "direct acting" machinery, and the third and fourth are of the crank-and-fly-wheel type. The figures do not include anything for buildings, land, chimneys, wells, boilers, etc.

The cost of boilers with mechanical stokers, feed-pumps and appurtenances, steam piping, and minor details—everything ready for service under average conditions—would be covered by $20 per boiler-horse-power.

It is impossible to include all plants, therefore these averages are based on:

Total water load against the plungers, 90 lb. per sq. in., or a head of 207 ft., including suction and friction;

Actual evaporation in the boilers under working conditions, 8 lb. of water per pound of coal, with feed at 150° and with coal at $3 per net ton of 2 000 lb.

Steam pressure at throttle valve of engine, 75 lb. gauge, for low-duty compound; 125 lb. gauge, for low-duty triple and cross-compound; 150 lb. gauge, for high-duty triple; an allowance of 5 lb. above the pressures given for boiler pressures.

The desire is often expressed for a schedule, rate of cost, or price of pumping engines, but it is a very difficult matter to make a price list at any certain time, which will be reliable beyond an approximate guide for estimate. Although the water-works pumping engine has been brought largely to a commercial basis in manufacture and sale, the conditions under which it must operate are special for the location where wanted, and all prices pertaining to specially defined contracts are more or less changeable.

About January, 1899, prices of all sorts of material began to rise, and by 1901 were at a higher point than for several years. In the spring of 1904 a downward tendency developed, and coke ovens in Western Pennsylvania were shut down. Pig iron began to decline, Portland cement fell off decidedly in price, steel products were lower, and cast-iron water pipe was at $23 per ton; but, in the autumn of the same year (1904); the drooping markets again strengthened, and advanced steadily until the summer of 1907, when cast-iron pipe was up to $34 per ton, steel products were high and hard to get, and orders were booked for a year ahead.

In 1900, at an attempted letting, the City of Cleveland rejected all bids for pumping machinery because the prices asked were prohibitive. The needed pumping engines were bought two years later at much better figures for the city. Based on the low figures of 1896, the record of prices for pumping machinery varied as follows:

(*A*) Bottom figures of 1896 represented by....... 100%
(*B*) Later figures............................ 137%
(*C*) Still later figures........................ 118%
(*D*) Figures in the summer of 1907............. 155%
(*E*) Comparatively recent figures............... 115%

Thus it will be observed that the best attempt can only result in a fair average, according to the labor and material markets, and also depending on the state of the shops bidding on the special work in question. When a shop is "hungry for work," bids will be low; but when it is "full up with work," bids will be high. All shops do not strike the tide at the same stage and at the same time, hence there is a certain amount of "irregularity" in the market, with a strong or weak undertone, as the case may be.

The higher steam pressures, which go hand in hand with greater steam economy, also changed ideas on boilers, brought greater horsepower per boiler by enlarging the units, and led to restricting the dimensions of the boiler plant. In the long run, and among the many plants built, the boiler for high pressure, made up of small diameters of parts, on the unit principle, economizes space, buildings, initial cost in large powers, and other important items, to a very satisfactory degree. Under present circumstances of unit capacity, cost of construction, convenience and economy of operation, together with

considerations as to buildings and space required, the water-tube boiler, fitted with automatic stokers and damper regulators, takes the lead as a general steam generator for water-works pumping plants. The practical relation between the economic duties of pumping engines and the amount of boiler power required, may be illustrated by Table 1, the measurable amount of boiler power being indicated by taking 10 sq. ft. of heating surface for each boiler-horse-power.

TABLE 1.—Boiler-Horse-Power Required for Each Pump-Horse-Power, Counting 10 Sq. Ft. of Heating Surface per Boiler-Horse-Power.

Duty in foot-pounds per 1 000 lb. of dry steam.	Boiler-horse-power per pump-horse-power.	Duty in foot-pounds per 1 000 lb. of dry steam.	Boiler-horse-power per pump-horse-power.	Duty in foot-pounds per 1 000 lb. of dry steam.	Boiler-horse-power per pump-horse-power.
40 000 000	1.63	120 000 000	0.55	165 000 000	0.40
50 000 000	1.32	125 000 000	0.52	170 000 000	0.39
60 000 000	1.10	130 000 000	0.51	175 000 000	0.38
70 000 000	0.94	135 000 000	0.49	180 000 000	0.37
80 000 000	0.83	140 000 000	0.47	185 000 000	0.36
90 000 000	0.74	145 000 000	0.46	190 000 000	0.35
100 000 000	0.66	150 000 000	0.44	195 000 000	0.34
110 000 000	0.60	155 000 000	0.43	200 000 000	0.33
115 000 000	0.57	160 000 000	0.41		

Table 1 is based on the fact, repeatedly demonstrated, that in good ordinary practice, 1 sq. ft. of heating surface will evaporate 3 lb. of water per hour, from 150° (temperature of feed), into steam at 150 lb. gauge pressure. Therefore 10 sq. ft. will evaporate 30 lb. of water as above, and this amount of evaporation is taken as 1 boiler-horse-power.

This is a good basis, and safe in most cases; but, if at any time caution or any special reason should suggest an increase in heating surface, any desired percentage of increase may be readily added without disturbing the relations of the different rates of economy. For example: If it should be decided that 2½ lb. of water per sq. ft. of heating surface per hour is all that it would be safe to reckon on, then 20% added to the boiler-horse-power of Table 1 would provide for such a case. Or, to go to extremes, if it was thought that 2 lb. of water per sq. ft. of heating surface per hour was the limit, 50% added to the figures in Table 1 would meet the demands for boiler capacity. With boilers properly constructed and arranged, however, Table 1 will answer all reasonable purposes.

The limits of steam economy in the pumping engine are about reached, both theoretically and practically. The modern duty record is shown by Table 2.

TABLE 2.—DUTY RECORD PER 1 000 LB. OF DRY SATURATED STEAM.

Year.	Duty, in foot-pounds.	Year.	Duty, in foot-pounds.
1893	154 048 700	1900	178 497 000
1895	157 843 000	1900	179 419 600
1898	167 800 000	1906	181 068 605
1900	168 532 800		

The leading type is a vertical, triple-expansion, condensing engine, with outside-packed plungers, mostly of the crank-and-fly-wheel type.

In the early days of the pumping engine the tendency toward increase in size was not realized, nor what tremendous capacities the pumping units would reach; but the view is clearer now, and there is no good reason for straying too far afield in search of the appropriate line to follow. It is not easy to forecast what developments may take place; but, with the analysis of coal, a full knowledge of the heat possibilities demanded by the steam at the working pressure, the efficiency of the boilers, a clear idea of the laws of steam expansion, and with the types of the most efficient pumping engines practically fixed, it is evident that some radical departure must be made to defeat what is known at the present time.

Table 3 gives an example of how methodically the cost of plants built on the unit basis may be determined. In some cases these figures may be too high and in others too low; they are closely approximate, and enough of the data are based on records fairly to insure the figures in the table as safe for practical use in making estimates. However, the table is so close that it would be taking chances for an engineer or a contractor to guarantee the production of results for the figures named, without investigating each case by itself. The work contemplated is for the best type of modern, triple-expansion pumping engines, and high-pressure boilers. The buildings are assumed to be of good design and quality; of brick, or of stone where stone is cheap; the roofs steel-trussed and slate-covered; the chimneys adequate; and the intakes properly proportioned and thoroughly screened. The cost given includes everything except the land.

TABLE 3.—COST OF COMPLETE PUMPING STATIONS.

Pressure of water load pumped against, in pounds per square inch.	Cost of plant, per million gallons capacity, including reserve.	Pressure of water load pumped against, in pounds per square inch.	Cost of plant, per million gallons capacity, including reserve.
30	$6 750	90	$8 250
40	7 000	100	8 500
50	7 250	110	8 750
60	7 500	120	9 000
70	7 750	130	10 000
80	8 000		

There are cheaper classes of pumping engines, but they are necessarily of lower economic efficiency, and therefore require more boiler capacity, more coal storage, and other incidentals which, when balanced up, will tend to keep the figures about the same. A cheaper and less durable building may be used, but in the long run this will need more repairs, which when capitalized will bring the account fully up to the figures given and most likely exceed them.

It is scarcely possible that the cost of equipping pumping stations for water-works will be increased much on account of a higher type of steam machinery, because it is evident that the top limit has just about been reached, with the record a little more than 181 000 000 ft-lb. per 1 000 lb. of steam. Ten years ago it nearly touched the 180 000 000 mark; and a gain of 0.8 of 1% in ten years, with every nerve strained, is eloquent evidence of the top limit. The Mariotte curve is about the nearest approach to perfection possible for the steam engine to accomplish, in expressing the relation between the work done and the amount of steam used. If the terminal pressure is taken as expressing the steam used, and all the steam is accounted for by the diagram, then somewhere in the immediate neighborhood of 180 000 000 duty, with 96% mechanical efficiency of the machine, will be the resulting figures, with a reasonable amount of steam used in the jackets and re-heaters charged against the account.

If there were no necessity for the use of steam-jackets, or jacket steam, the figures would approach 200 000 000 rather closely, and if superheating can save jacket steam, and vitalize the working steam in the cylinders, the latter figure may be reached in the near future, as far as the official test is concerned. This pleasing result may have to be obtained, however, by the use of a surface condenser with a com-

paratively small air-pump, and this type of condenser may require more maintenance account than the jet form; and the superheat may have to be obtained at the cost of coal.

If the steam turbine can step forward at this stage of the performance, and show something better in the matter of water sent up the hill in proportion to expense incurred, it is now the proper time to do so. This would effect something of a change in the pumping station, although as far as present evidence goes, the change will be in the direction of smaller engine-houses and larger boiler- and coal-houses. The only claim in sight for the steam turbine on pump work is a lower first cost, it being evident, thus far at least, that it cannot compete in steam economy with that form of the reciprocating engine used in pumping water for public supply. Whether the cost of construction of the steam turbine and centrifugal pump plant will be materially less than that of a reciprocating plant, when all difficulties are overcome and the turbine machine is brought to what it must eventually be, time only will tell. The writer at present is carefully investigating the turbine proposition and its possibilities as a practical water-works factor, but is not ready yet to declare results.

Unless something develops which is not yet in view, the ideal turbine pumping engine will be a steam turbine driving a centrifugal pump, all mounted on the same frame or bed-plate. There are conflicting conditions accompanying the combination of the steam and water turbine in the same machine, and it is very much the same old story learned by the reciprocating engine—the demands of a highly elastic fluid at one end of the machine and a very stubborn and practically non-elastic fluid at the other. No reciprocating steam engine can live or give good economy under the high rotative speed required for direct connection to centrifugal pumps under the ordinary heads or pressures demanded by the average water-works, so that a direct-connected reciprocating steam end, and a centrifugal water end are practically impossible. The direct-connected steam and water turbine has been used on a small scale, with doubtful economy, and is being attempted on a large scale where fuel is cheap, with what all-around success remains to be seen. It looks like the old controversy between low first cost with large fuel bills, and higher first cost with low fuel bills.

Plants equally constructed and with equally low repair and maintenance accounts can have no advantage over each other excepting in the matter of fuel economy; and the lessened quantity of coal to be bought is the real foundation on which to base an increased investment in the plant. There are only two important items which grow less by higher duty, and these are the coal account and the fixed charges on the boiler plant. Everything else increases with higher duty, excepting the wages account for equal capacities, and this remains at least as much with high as with low duty, excepting with the large high-duty triple engines, and with these the wages are somewhat less in the fire-room on account of the lessened quantity of coal to be shoveled in proportion to the pumping.

The items for and against the high-duty pumping plant are:

Against High Duty:	In Favor of High Duty:
Maintenance account for machinery.	Maintenance account for boilers.
Interest on machinery.	Interest on boilers.
Oil, waste, packing, etc.	Sinking fund for boilers.
Sinking fund for machinery.	The coal account.

To ascertain which type and class it will pay best to buy, requires comparisons and calculations which call for thought and care, but which may be readily enough made when the conditions of the contemplated plant are clearly laid down. The fixed charges against pumping machinery and boilers are as follows:

Vertical Triple-Expansion Pumping Engines:

Maintenance account	2%
Interest account	4%
Sinking fund account	3%
Oil, waste, packing, and small repairs	1%
Total fixed charges	10%

All Other Types of Pumping Engines:

Maintenance account	3%
Interest account	4%
Sinking fund account	5%
Oil, waste, packing, and small repairs	1%
Total fixed charges	13%

The reason for the difference of 2% in the sinking fund account, between vertical triple machinery and all other forms, is that it has become evident that the vertical triple type of pumping engine will probably not be replaced as obsolete by any of the other types, but the other types will eventually give way to the vertical triple. Under these circumstances, when the vertical triple is properly built, its life is taken at 33⅓ years, and that of all the other types at 20 years; representing, respectively, 3% and 5%, to make up the sinking fund to 100% in the time specified.

The fixed charges against the boiler plant are as follows:

Maintenance account	5%
Interest account	4%
Sinking fund account	5%
Total fixed charges	14%

Even when the cost of coal is $1 350 per year for each million gallons of water pumped per day, the total coal bills do not amount to very much for 2 000 000 or 3 000 000 gal. per day; but when the quantity gets toward 10 000 000 gal., the figures are more important. The larger triple-expansion pumping engines of the reciprocating, displacement type, pump 1 000 000 gal. per day with $625 worth of coal per year for the best records, and $900 is an ordinary good record; while $1 350 is about the best steam-turbine pumping record, as far as the writer can learn. It looks as though the great struggle to keep down the capital account in the turbine outfit must be partly abandoned, as far as water-works service is concerned.

If a high peripheral velocity is what the steam turbine needs, it may be necessary to increase its diameter to meet this demand, and thus reduce the rate of revolution to meet the demands of the centrifugal pump. To obtain the greatest possible effect in the steam turbine, the expansion and velocity of discharge of the steam, would require a velocity of periphery quite beyond practical limits, as centrifugal force interferes in this direction. On the other hand, the peripheral velocity of the runner of the centrifugal pump should be somewhere in the neighborhood of the velocity of a jet of water under the pumping head, represented by the well-known formula $V = \sqrt{2\,gh}$.

These are two conflicting conditions which must be met if any material economy is to be obtained by this form of pumping apparatus. Attempts to compromise are frequently made by accommodating the steam turbine work, and then reducing the diameter of the pump runner to keep down its rim speed to something like the equivalent velocity of falling bodies from the height represented by the water head pumped against. This practice results in a direct loss of mechanical efficiency in the smaller pump as against a larger pump at a lower rate of rotative speed.

Within its limitations, the centrifugal pump is an ideal machine for lifting water; which means that it must be designed and constructed for known positive conditions, if the best results are looked for. At constant speed, for a considerable range of capacity, as is often found in water-works, or for a varying head—very frequently found in the Middle West, where domestic and fire service are combined in the same apparatus—it is inefficient and unreliable. Even on stand-pipe work, as at Schenectady, N. Y., the centrifugal pump proved to be a total failure, and that city is now seriously contemplating the construction of a reservoir for the relief of these pumps; and there are others.

The centrifugal pump sends the water by centrifugal force, from the center toward the outside, and the main principles to be followed in designing are the prevention of:

(*a*) Sudden changes in water velocity,
(*b*) Sharp changes in direction of flow,
(*c*) Unnecessary friction losses.

With a steep vane there is required a less number of revolutions per minute; with a flat vane there is required a higher number of revolutions per minute, to do the same work. The above-mentioned formula ($V = \sqrt{2\ gh}$) gives an average result in feet per second of the peripheral speed of the runner; the actual results depend on the angle of the vanes and the width of the impeller.

A case in practice, which has just come to hand—the figures being taken from the annual report of the operation of a water-works plant—will be to the point, in comparisons between displacement and centrifugal pumps in practical water-works service. The data are as follows:

Pumpage per day..............	9 000 000 U. S. gal.
Head against displacement pumps..	207 ft.
Head against centrifugal pumps..	23 ft.
Cost of centrifugal pumps........	$10 000.00
Cost of displacement pumps......	50 000.00
Cost of fuel used for both plants...	1.25 per net ton.

COST FOR FUEL TO PUMP 1 000 000 GAL. 207 FT. HIGH.

With displacement pumps......................	$3.27
With centrifugal pumps........................	6.48
Difference in favor of displacement pumps..	$3.21

The difference in cost of fuel per annum at 9 000 000 gal. daily pumpage, against 207 ft. head, in favor of displacement pumps, amounts to $10 544, which means that the economic duty of the displacement machinery is 1.96 times that of the centrifugal, or the ratio between a cost per million gallons 207 ft. high, of $6.48 and $3.27, as shown by the coal account given. It may be noted in passing that the mechanical efficiency of the displacement machinery is 93%, and that of the centrifugal machinery is 65%, which corresponds with the results found in general practice along these lines.

The gas engine as a water-works pumping power is entirely undeveloped on anything like the scale which it will have to reach to be seriously considered in fairly large schemes of pumpage. There will be found to be a great variation in gas production from different kinds of coal, and there is likely to be as much variation as in the generation of steam from different fuels. As a practical fact, the power developed by gas engine cylinders will be found to lessen at considerable elevations above sea level, in a certain proportion to diameter of cylinder, on account of the diminished atmospheric pressure and consequent diminished quantity of oxygen per unit of air. For example: Gas engines of 100 b.h.p. are scaled down to 80 h.p., with a fixed size of cylinder, under guaranties made for Denver, as against the operation of the same engine at or near sea level.

Developments along the line of gas-power pumping for waterworks are going on, and good results in economical and reliable gas production, easy of manipulation, will be one of the improvements in the future not very far away. The large gas engine available for

water-works pumping has not yet come to the front, and there are certain adaptabilities necessary to be made—in regard to speed, for example—which complicates the problem more or less. There are now many small plants for pumping, where the gas engine fits in very well, but there are some incidental items aside from fuel economy which have weight in small plants, but which would not be considered in a large pumping plant. The small steam pumping plant is extravagant of fuel, while the small gas plant is practically as economical as the larger ones, and although the application of steam power to pumping is more direct than with gas, the great fuel economy of the gas-power apparatus enables it to operate at a profit at a much lower mechanical efficiency than the steam machine. For example: A small gas engine geared to a triplex power pump may result in a total mechanical efficiency of 75%; then, if the engine and producer give out a brake-horse-power for 1½ lb. of coal per hour, the coal per pump-horse-power will be 2 lb. The small steam pumping plant with a mechanical efficiency of 94% requires not less than 4 lb. of coal per pump-horse-power-hour, and it is doing pretty well to accomplish this.

The indicated power with steam and the brake power with gas may be taken on equal terms in the larger plants, but the direct application of power with steam and the indirect application with gas, as expressed by a mechanical efficiency of 95% for steam and 75% for gas, shows why the gas cannot compete in the larger units in the present-day pumping plant until some more direct application of its power to pumping can be made. In the smaller and more wasteful steam plants, although the mechanical efficiency holds up very high, the steam economy is low, for the reason that the smaller quantity of water will not justify the cost and complication which go with extreme high duty, but which pay on a larger scale. Under these circumstances, the gas plant shows to advantage because its coal economy is just about as good as in the larger units under practically a uniform mechanical efficiency.

The mere recording of various statements of economy in coal per power unit does not tell the entire story, as will be shown by the analysis of data. For example: Confining the comparison for the moment to steam pumping plants, the writer has two records; one of 1.02 lb. of coal per indicated-horse-power-hour, and the other 1.98 lb. Both records were obtained as nearly as could be under similar conditions in actual water-works pumping. The fuel calling for 1.98 lb. is

slack coal at $1.50 per net ton; and the fuel calling for 1.02 lb. is anthracite coal at $4.50 per net ton. The analysis of the two fuels for heat units on complete combustion is as follows:

Heat units developed by the slack........ 11 000 per lb.
Heat units developed by the coal......... 14 000 " "

In the case of the slack coal, the buyer obtained 146 000 heat units for 1 cent. In the case of the anthracite coal the buyer obtained 62 000 heat units for 1 cent. The plant consuming the slack coal used 363 heat units per horse-power-minute, while the plant with the anthracite coal consumed 238 heat units per horse-power-minute. The efficiency of the boilers was 70% with the slack and 80% with the anthracite. This, reduced to work demonstrated by indicated horse power, gives 13 200 000 ft-lb. for 1 cent for the anthracite and 9 805 680 ft-lb. for 1 cent with the slack, with the efficiencies of the boilers equalized; and this shows that a difference of 3 394 320 ft-lb. for 1 cent existed in the difference in steam economy of the two engines. The engine with the slack fuel gave 130 000 000 ft-lb. per million B.t.u., and that with the coal gave 163 000 000 ft-lb. per million B.t.u., further demonstrating the difference in steam economy in the two machines. The gain in foot-pounds for 1 cent with the engine using anthracite coal over the other is 34%, and the gain on the heat-unit basis in favor of the anthracite coal engine is 26%, which indicates good judgment in putting enough investment into the engine, boilers, and fuel to obtain good results. The engine with the coal fuel cost $4 166 per million gallons capacity per 24 hours; and the engine with the slack fuel cost $4 083 per million gallons capacity per 24 hours. This price per million gallons does not include foundations and appurtenances, so that the $4 800 per million gallons for the vertical triple pumping engine of large size given earlier in this paper is fairly well checked, as the latter figure includes everything in the engine-house belonging to the machinery. It may be of some interest to know that the two pumping engines which have just been compared were by different builders.

The present high type of water-works pumping engine apparently cannot be improved to any material extent in principle, so that the very best that can be expected is an extremely small increase in economy secured by improved construction. It is very probable that

an increase in fuel economy from 6% to 12% can be gained in many steam plants by the use of superheated steam. Some think this too low a figure for the gain by superheat, but it must not be forgotten that many of the good records for superheat have been made where engines and boilers of only fairly good economy are in use, whereas, when the best type of pumping engine is supplied with superheated steam, the application is then made to steam machinery of the very highest class, where all the steam refinements can be operated to the best advantage, and where multiple expansion, steam-jackets, re-heating, etc., are used under the very best conditions to reduce internal losses and condensation to an extremely low point. A pumping engine at the Boston high-service station at Chestnut Hill Reservoir has a capacity of 30 000 000 U. S. gal. per 24 hours, and has the following data to its credit:

Duty per 1 000 lb. of dry saturated steam (no superheat)	178 497 000 ft-lb.
Duty per 1 000 000 B.t.u.	163 925 000 " "
Duty per 100 lb. of coal	173 869 000 " "
Dry saturated steam per indicated horse power per hour	10.34 lb.
Coal per indicated horse power per hour	1.06 "
British thermal units per indicated horse power per minute	188

If the duty for this engine, with dry saturated steam, should be increased 12% by superheating, it will read 199 916 640 ft-lb., which is about 2 000 000 above the highest record with the use of superheated steam. The highest known duty per 1 000 lb. of dry saturated steam —in round numbers 181 000 000 ft-lb.—would go up to 202 720 000 ft-lb. with 12% increase, which is considerably above the superheat results thus far obtained.

In the present-day pumping engine, the steam-jacket system has been modified somewhat from the earlier practice, and brought to a high state of efficiency by keeping the heat in the jackets close to the balancing point, where the greatest good will be done with the least loss from useless internal radiation. The distribution of the steam is now very satisfactory, and the waste room or clearance at the

cylinder ends is brought down to surprisingly low terms by the better arrangement of valve gear and steam ports.

If there is any waste heat in the smoke-flues or uptake of the boilers, re-heaters for the receiver steam can be provided, and this steam can be made a vehicle for the transportation of heat now getting away up the chimney back to the engine, and there made to do work. This is no special credit to the engine, beyond its providing facilities for the use of such heat as the boilers are allowing to escape; but, in the practical operation of the plant, it will reduce the coal account by turning into useful work some of the heat of combustion not absorbed by the boiler heating surfaces. It is not entirely clear why more of this practice of flue re-heating has not been done; it is certainly an old enough idea, long known. In the Pawtucket pumping engine, a small cross-compound, built by George H. Corliss in 1878, flue re-heating was successfully used. This engine gave a duty of 133 000 000 ft-lb. per 100 lb. of coal.

The quadruple-expansion pumping engine, with its additional steam cylinder, resulting either in a tandem arrangement, or an abandonment of the three-plunger design so favorable to uniform hydraulic effects, has no great future before it, and will probably never come into water-works service. With the 33 to 40 steam expansions practicable with triple machinery possessing such admirable mechanical lines, and with the conversion of heat into work about up to the practical limits, there seems to be no room for the quadruple-expansion engine. The only prominent attempt in this direction thus far made found its way to the scrap heap after a short lease of life.

Some, who are not familiar with pumping engines at work, rebel at the idea of triple-expansion engines with aims at economy; but there are other items, aside from the particular method of handling the steam, and among them the fact that the distribution of water into a force main from three plungers has never been surpassed, and probably has never been equalled. In addition to that fact, the triple-expansion pumping engine holds a record for economy of steam far ahead of compound machinery. In nearly all cases a compound would have to be fitted with two double-acting plungers on account of pulsations in the mains; horizontal engines would be rather limited in capacity, and a cross-compound, vertical machine would be objectionable to a very serious extent.

In 1878 the direct-acting, non-rotative, compound, condensing pumping engine held the front line, and successfully disputed the field with the several odd and unrepeated forms of the crank-and-fly-wheel machine which at that time aimed at high steam economy. For years the direct-acting engine, as an all-around better proposition, held sway, and its durability and reliability as a machine for pumping water for public supply have never been excelled. Gradually, however, the financial end of the crank-and-fly-wheel manufacturing establishment and the engineering end drew closer. Then the direct-acting engine took on a high-duty attachment to increase its steam economy as it became more and more hard pressed in the fight of low duty against high duty. The problem in the crank-and-fly-wheel camp was to reduce the capital account by simplifying the design, and keep on raising the duty.

The growth and drift of the public pumping engine seems to be toward larger and larger units, and the introduction of the higher duty idea into smaller and smaller units, with the vertical, triple-expansion engine rapidly taking the lead, and the cross-compound crank engine gradually forcing the direct-acting machine out of the field in the moderate and small sizes. The direct-acting, non-rotative, triple, with six steam cylinders and two double-acting plungers, stubbornly holds on, and probably will for a while longer. Just what the next ten years will bring out is not known. It looks doubtful for the dominance of the turbine type in that time if at all, and the gas engine has not yet really made a good start for the front line. Superheated steam and generally higher steam pressures up to about 175 lb. gauge pressure will take place gradually, and that will be the last firing line of the reciprocating, triple-expansion, displacement, pumping engine. It will hold this line stubbornly, and it will require a great deal more progress than is evident in any direction at present to dislodge it or even to shake it materially. Its capital and fuel accounts, even with coal at a moderate price, make a satisfactory showing, and its maintenance in the presence of good design and workmanship will not exceed 2% in large or moderate plants where it is properly cared for.

The perfect present-day pumping plant involves the following items:

Vertical, triple-expansion, crank-and-fly-wheel pumping engines;

Long stroke, with rotative speed not to exceed 20 rev. per min.;
Maximum piston travel 200 ft. per min.;
Modified steam-jacketing and re-heating;
Steam pressure at throttle, 175 lb. per gauge;
Moderately superheated steam by independent apparatus;
Smoke-flue re-heating;
Water-tube boilers;
Mechanical stokers;
Natural draft at least 0.8 in. of water;
Feed-water economizers;
Automatic damper regulators;
Coal bought on the basis of 14 000 heat units per lb.;
Boiler efficiency of 75%;
Coal per indicated horse power, 1 lb. for large plants;
Coal per indicated horse power, 1.75 lb. for small plants;
Maintenance of engines, 1.5% for large plants;
Maintenance of engines, 3% for small plants.

These and other items of a similar nature are about what a look ahead discerns as the coming events in the planning, construction, and operation of present-day pumping plants for public water supply.

Chicago, St. Louis, Cincinnati, New Orleans, Cleveland, Buffalo, Pittsburg, Philadelphia, and other cities, large and small, will have to depend on pumping for the main supply; and New York, Boston, Baltimore, and other cities, in which the growth extends to levels too high for distribution from the general source of supply, will have to use auxiliary service involving pumpage for high-service supply. Therefore, it will be noted that the use of the best present ideas, and the introduction of new ones, will be in order for a long time to come, in the municipal water-supply pumping station.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

STREET PAVING CROWNS, WASHINGTON, D. C.*

By T. J. Powell, Assoc. M. Am. Soc. C. E.

The writer has read with much interest the paper by P. E. Green, Assoc. M. Am. Soc. C. E., entitled "A Review of Chicago Paving Practice,"† and also other recent papers‡ on this subject, and submits the following:

There seems to be a great difference of opinion among engineers on this subject, and, as Mr. Zahniser says, a collection of these formulas may be the beginning of a possible "Rational Formula."

The following method of treating crowns has been used by the Engineer Department of the District of Columbia since 1894, but has only recently been formulated.

This formula was suggested and deduced by Mr. Joseph W. Dare, Assistant Engineer of the District of Columbia, and takes into account the width of the roadway and the longitudinal grade of the street. It is applicable for all widths of roadway up to and including 50 ft., after which it is necessary to treat the section as a special one The formula is:

$$C = \frac{W\,(100 - 4\,P)}{6\,300 + 50\,P^2}$$

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

† *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 1.

‡ In *Engineering News*, by Mr. George C. Warren, entitled "Paving Practice with Respect to Crowns of Roadway Pavements and Concrete Foundations," December 2d, 1909; by Mr. G. B. Zahniser, entitled "Suggestions for a Rational Formula for Street Pavement Crowns," May 5th, 1910, and by James N. Hazlehurst, M. Am. Soc. C. E., entitled "Formulas for Determining Street Crowns when the Two Curbs are at Different Elevations," June 30th, 1910.

In which C = the crown, in inches,

P = the longitudinal grade, expressed as a percentage,

W = the width of the roadway, in inches.

When the curbs are level, the crown is distributed as shown by the following formula:

$$\frac{8\,C}{0.3\,R} = d$$

$$a \text{ or } b + \frac{C}{18} = \text{the elevation at } A \text{ or } D;$$

$$a \text{ or } b + \frac{C}{12} = \text{the elevation at } B;$$

d = the transverse grade, expressed as a percentage;

a or b = the elevation at the gutters, expressed in feet and hundredths.

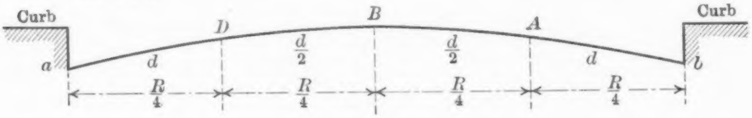

FIG. 1.

As will be noticed in this formula, the percentage of grade from gutter to quarter and from quarter to crown in no case exceeds 4% and 2%, respectively, and the diagonal rate from quarter to the curb, along the hypotenuse of an isosceles triangle, the legs of which are equal to the distance from the quarter to the curb, will not materially exceed the longitudinal rate, thus keeping a team on the same rate of grade, whether they are going straight up hill or taking a diagonal course as is their desire if left to themselves.

This, as far as the writer knows, is the only formula in which this holds good.

Another point, which has been taken into consideration and is shown in the following formulas, is the location of the crown when there is a difference in the elevations of the curbs.

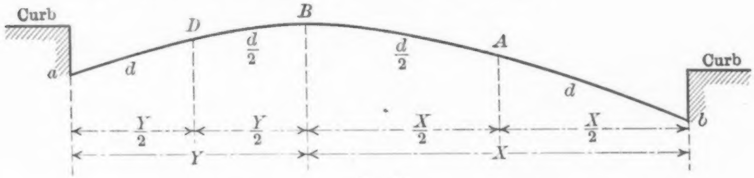

FIG. 2.

$$\frac{a - b}{1\frac{1}{2}\,d} + \frac{R}{2} = X$$

$$R - X = Y$$

$$b + \frac{X}{2}\,d = \text{the elevation at } A;$$

$$b + \frac{3\,X}{4}\,d = \text{the elevation at } B;$$

$$a + \frac{3\,Y}{4}\,d = \text{the elevation at } B;$$

$$a + \frac{Y}{2}\,d = \text{the elevation at } D;$$

X = the long side of the crown;
Y = the short side of the crown;
R = the width of the roadway, in feet and hundredths;
d = the transverse grade, expressed as a percentage.

This formula puts the crown and quarter points in such a position that the transverse grades from gutter to quarter and from quarter to crown will be the same as if the curbs were level, for the same longitudinal grade.

It is the practice in Washington, D. C., to put vitrified block gutters on streets on which the longitudinal grade is 1.5% or less. This does not change the formula, as 0.1 ft. is added to the curb side of the gutters for the rise of the same. The crown is then worked, using the distance between gutters as the width of roadway.

These formulas refer particularly to streets paved with sheet-asphalt, but for asphalt block, granite block, or other pavements having a more or less rough surface, and therefore giving a more secure foothold, it can be used equally well and give as good results by the addition of 1 in. to the amount of crown given by the above, This gives rates a little steeper from gutter to quarter and from quarter to crown, which is necessary, as these materials when paved on a gravel or sand base, as is usually the case, have a tendency to settle until all joints are entirely closed and water does not get between them.

These crowns give transverse grades sufficient to carry all water to the gutters as rapidly as necessary, besides reducing to a minimum the number of accidents caused by horses falling; and it produces a section which is pleasing to the eye.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SINKING A WET SHAFT.

By John P. Hogan, Jun. Am. Soc. C. E.

To be Presented May 3d, 1911.

On the new Catskill Aqueduct for New York City some of the most difficult problems, both in design and construction, have been encountered in the deep pressure tunnels or siphons. The design and location of these tunnels have been described in the technical press.* The first siphon to be placed under contract was that crossing the Rondout Valley, near High Falls, N. Y., a work second in importance only to the Hudson River crossing, and now more than half completed.†

There are eight shafts on the Rondout Siphon, varying in depth from 374 to 710 ft. (three permanent shafts and five sunk to expedite the construction of the tunnel), and all, excepting Shaft No. 4, were completed to tunnel grade without any great difficulty. It will be noted on the profile, Fig. 1 and Plate XXVI, that the tunnel between Shafts Nos. 3 and 5 passes under Rondout Creek and its buried preglacial gorge, and, due to a sharp steepening of the dip, also passes through a number of different rock strata. The examination of exposures of these rocks at other points (and preliminary surveys and borings) indicated that some of them were very hard while others were of a porous, water-bearing nature. It was evident from the start,

* *Engineering Record*, January 29th and February 5th, 1910; and *Proceedings*, Municipal Engineers of New York City, 1909.

† A description of the power-plant for this work was published in *Engineering Record*, April 10th, 1909; and a description of the methods of tunnel excavation on January 1st, 1910.

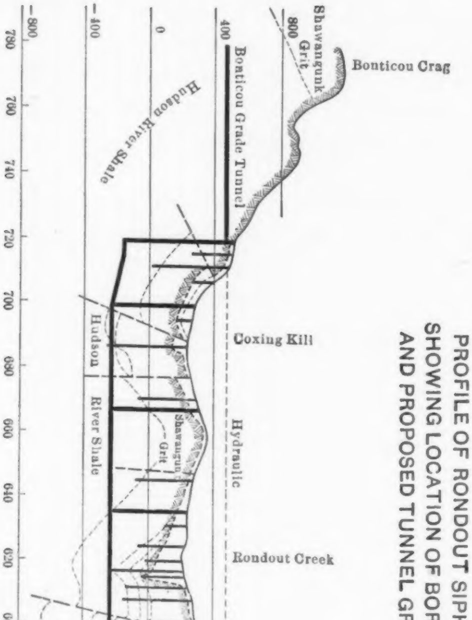

PROFILE OF RONDOUT SIPHON
SHOWING LOCATION OF BORINGS
AND PROPOSED TUNNEL GRADE.
Solid lines show borings, thus
Bonticou Crag
Shawangunk Grit
Bonticou Grade Tunnel
Hudson River Shale
Coxing Kill
Hydraulic
Rondout Creek
Grade
Preglacial Gorge
Hudson River Shale
Shawangunk Grit
Esopus Shale
Helderberg
Limestone
Shawangunk Grit
Onondaga Limestone
Esopus Shale
Hamilton Shale
Cut & Cover
400
0
-400
-800

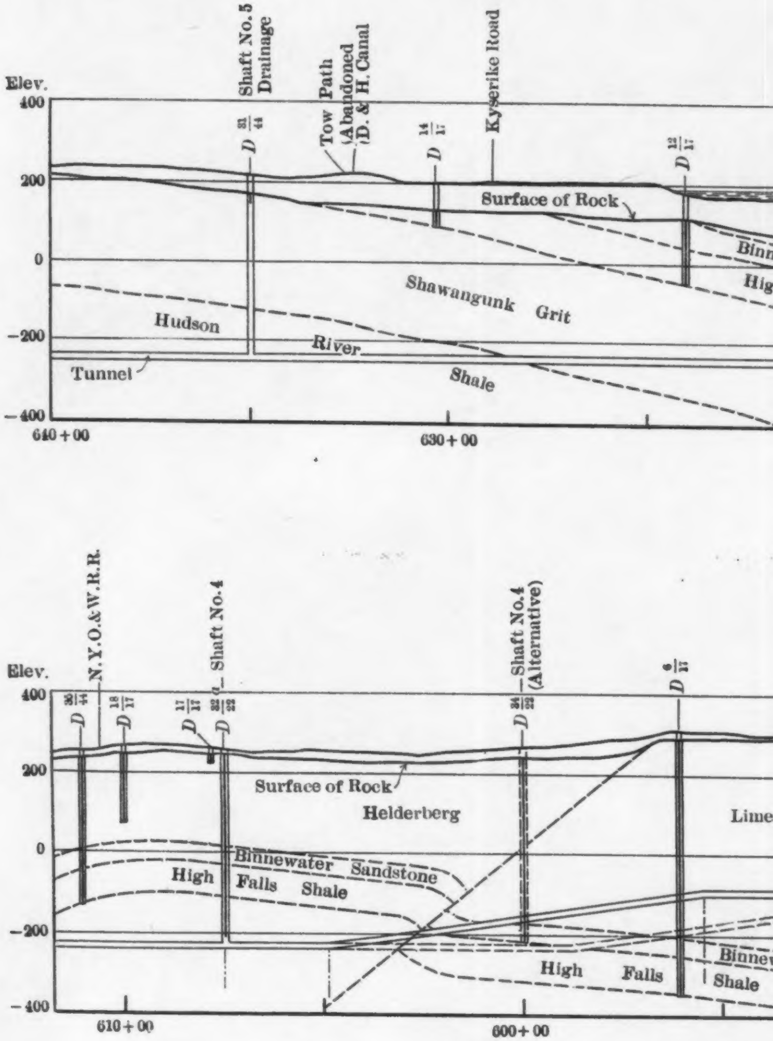
FIG. 1.

therefore, that this would be the most difficult part of the work, and that an intermediate shaft would be required in order that the progress might be kept equal to that in other parts of the tunnel. Any location of the shaft would require it to penetrate the strata in which the trouble was expected, and, for various reasons, the choice was limited to the stretch between Stations 600 + 00 and 607 + 50, as shown on Plate XXVI.

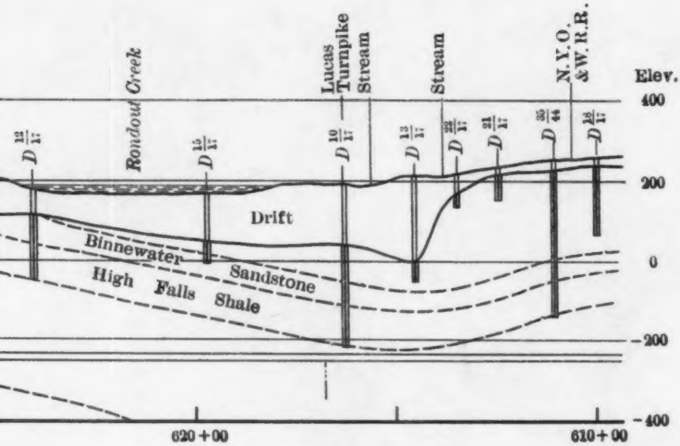
Diamond-drill borings were put down in the vicinity to define the strata, and at Stations 600 + 00 and 607 + 50, 4-in. shot-drill borings were made to permit of pumping experiments by which it was hoped to determine the relative porosity of the rock. An oil-well rig, having a capacity of about 90 gal. per min., was used in these experiments. The ground-water elevation was about 30 ft. below the surface at both holes. At Station 607 + 50 the discharge was about 50% in excess of the theoretical capacity of the pump, reaching a maximum of 130 gal. per min., and, although 1 071 000 gal. were pumped, it was impossible to lower the elevation of the ground-water. At Station 600 + 00 there was no excess in discharge over the capacity of the pump, and the ground-water level was lowered 53 ft. by pumping 2 020 000 gal. The surprising part of this experiment was that the ground-water level at Station 607 + 50 was also lowered 15 ft., although previous pumping at that point had had no effect.

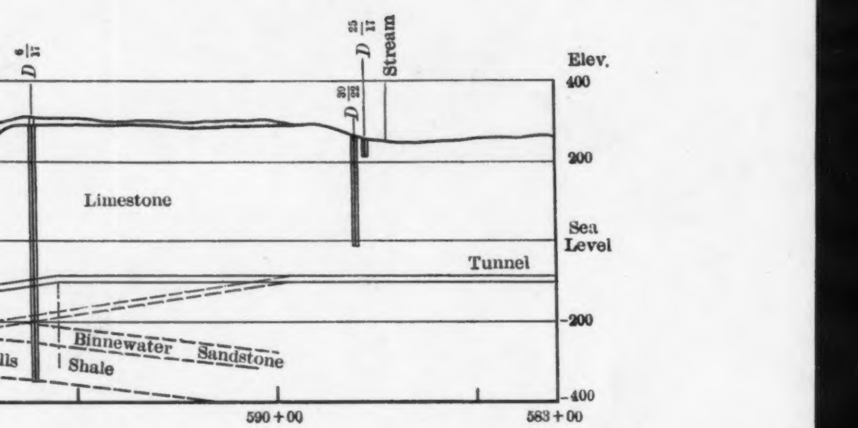
The water at both points was strongly impregnated with sulphur, and, while there was no thoroughly satisfactory explanation of these phenomena, the theory was advanced that at Station 607 + 50 gas was liberated by the churning action of the pump and thus increased the pressure, forcing the water through the working barrel of the pump. The records of the borings showed that the water was first encountered in the Binnewater sandstone. This entire bed is exposed in a vertical cliff below the High Falls, on Rondout Creek, less than a mile from the tunnel line, and examination showed it to be a soft, coarse-grained sandstone with numerous porous layers, probably caused by the solution of calcareous material. The High Falls shale immediately below it also showed considerable porosity, and the indications were that the shaft would be a wet one. The location at Station 600 + 00 seemed to promise a better shaft, but that at Station 607 + 50 gave a reduced length of tunnel in difficult rock. Both locations were shown in the contract drawings, and the contractor was given the option of choos-

PROFILE OF RONDOUT PRESSURE TUNNEL IN



UNNEL IN VICINITY OF SHAFT No. 4.





ing either one. The contractor elected to sink at Station 607 + 50. The result of the pumping experiments was to include in the contract provisions for an emergency pumping plant having a capacity of 1 800 gal. per min.

The total depth of the shaft is 498 ft. and the strata penetrated were: 6 ft. of glacial drift, 226 ft. of Helderberg limestone, 39 ft. of Binnewater sandstone, 92 ft. of High Falls shale, and 134 ft. of Shawangunk grit. The Helderberg limestone is a hard, dark gray rock, with layers of flinty material and few seams; the Binnewater sandstone is a soft, greenish, fine-grained rock of a high general porosity, and with numerous open water-bearing seams; the High Falls shale is a soft rock, thin-bedded in horizontal layers, with numerous seams, and varying in color from light green to red; the Shawangunk grit is an exceedingly hard, white, quartz conglomerate. The shale and sandstone grade into each other very gradually, the latter containing numerous shaly layers and *vice versa.*

The contract was let on June 12th, 1908, and excavation at Shaft No. 4 was started on July 20th, 1908, air being supplied from a temporary plant. Less than four months later, air was turned on from the large central plant.* The shaft, being for construction purposes only, is rectangular, 10 by 22 ft. over all. It is timbered throughout with 10 by 10-in. wall plates, 6 by 8-in. buntons and 2-in. lagging, making its size 8 by 20 ft. inside the timbers, and is divided into three compartments. Four No. 12 Tandem Cameron sinking pumps, each having a capacity of 450 gal. per min., and one No. 10 having a capacity of 300 gal. per min., were ordered for the emergency pumping plant, and, pending delivery, several No. 7 and No. 9 Cameron sinkers were kept in reserve.

The shaft was dry to a depth of 80 ft., but, on September 2d, the water rushed in from the bottom through the 4-in. bore-hole at an estimated rate of from 600 to 800 gal. per min., and the shaft was filled to within 40 ft. of the top. As the emergency plant had not then been delivered, a simple air-lift was installed which at first threw 1 600 gal. per min. As the efficiency decreased, due to lack of back pressure, two more stages were added, and, with the aid of two No. 9 sinking pumps, the water was lowered to within a few feet of the bottom. A 4-in. swedged nipple was then driven into the hole, and

* Described in *Engineering Record*, April 10th, 1909.

casing was attached. It was decided to grout the bore-hole, and a 1-in. pipe was let down inside the hole to the grit and extended to the surface. The shaft was then allowed to fill up, to equalize the pressure and prevent flow, and 1:1 grout was poured down the 1-in. pipe, while the latter was raised gradually. When the hole was filled to the top with grout, the 1-in. pipe was withdrawn, the shaft was unwatered, and sinking was resumed.

From this point to the bottom of the limestone the shaft was so dry that occasionally water had to be carried down to mud the drills. It was realized, however, that a large quantity of water under heavy pressure was present in the lower strata, and serious doubt began to be felt as to the possibility of sinking this shaft by ordinary methods. J. F. Sanborn, Assoc. M. Am. Soc. C. E., Division Engineer of the Peekskill Division, Northern Aqueduct Department, then directed attention to the recent successful use of the cementation or grouting process of sinking.* The contract called for grouting behind the lining of the completed tunnel, with a pressure of 300 lb. per sq. in., and the delivery of this apparatus was hastened.

At a depth of 215 ft. the shaft entered the Binnewater sandstone, and in this formation the water gradually increased until, at a depth of 260 ft., the shaft was making 225 gal. per min. This was handled by two No. 9 Cameron sinkers. While the sump was being drilled, on December 20th, 1908, an additional flow of about 600 gal. came in through one of the drill holes under considerable pressure, quickly drowned out the sinking pumps, and flooded the shaft to within 70 ft. of the top. Preparations had been made to plug any water-bearing hole, but the flow was of unexpected force and volume. It is interesting to note that the water, at this time and later when the shaft was flooded, rose to a lower elevation each time, indicating that the pumping had considerable effect on the general water-table.

A long delay now ensued before sinking was resumed. A single-stage lift, having an 8-in. discharge and 2-in. feed, lowered the water to within 60 ft. of the bottom, throwing from 600 to 350 gal. per min. A second stage of the same size was then installed, discharging into the first 100 ft. from the bottom. This had an initial capacity of 500 gal. per min., and lowered the water to within 30 ft. of the bottom. A third stage was added, and, with the aid of two No. 12 Cameron

* Described in the *American Engineering and Mining Journal*, August 1st, 1908.

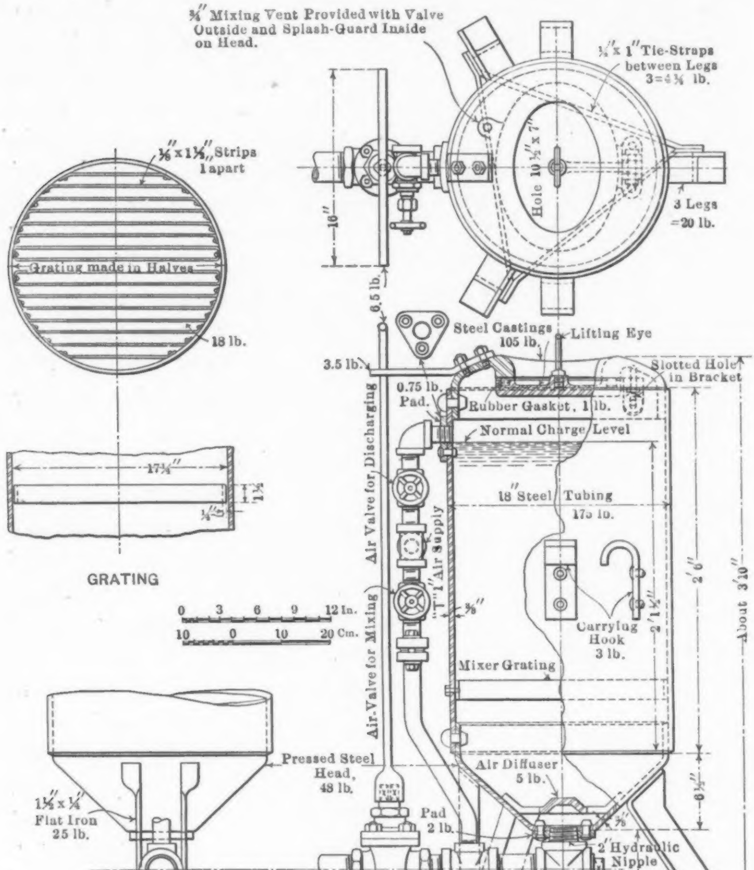
sinkers, each having a capacity of 450 gal. per min., the bottom was unwatered on January 24th, 1909, the No. 9 Cameron pumps were recovered, and the hole was plugged with a 2-in. swedged nipple and gate-valve. A gauge on the pipe indicated a pressure of 75 lb. Drilling was resumed for the purpose of finding the location and extent of the water-bearing seams, the holes being plugged with pipes and valves as water was encountered. On February 3d, 1909, while one of the pumps was temporarily out of commission, two of the holes struck water. The discharge hose broke on the second pump, when speeded up, and the shaft was again flooded and the pumps drowned. This was the beginning of a series of troubles with the pumps which will be described in detail later. Due to their great weight, the high head, and the consequent force of their pulsation, these pumps gave continual trouble with their discharge lines, and discharge and suction hose pipes. These troubles were minimized later by fastening them to the timbers and by improvements in the quality of the equipment, but the sinking pumps could never be relied on for satisfactory service on heads greater than 200 ft.

The pumps were recovered by using the air-lift and an additional No. 12 pump was lowered into the shaft. On February 10th, before the holes in the bottom could be plugged, the pumps again went out of commission in rapid succession, the shaft was again flooded, and the pumps were submerged. Again the pumps were recovered by the air-lift and the two No. 9 Camerons; the bottom was uncovered and the holes were plugged on February 16th.

Drilling in the bottom was resumed through pipes and gate-valves which could be closed quickly. The leakage from the sides and around the pipes in the bottom had now risen to 450 gal. per min. With the discharge hose and gaskets on the discharge line blowing out above, and water spurting in from the bottom, difficulty began to be experienced in getting labor. On February 21st the shaft was again flooded, due to pump troubles, and was not recovered by the air-lift until March 12th.

A sufficient number of holes had now been drilled to indicate that the rock for at least the next 8 ft. was soft and porous with several open seams of a maximum width of 8 in. The largest seam was at one point within 18 in. of the bottom of the shaft. The behavior of single 2-in. drill-holes made it unpleasant to contemplate the quantity

of water which might be encountered if sinking were resumed. Accordingly, it was decided to attempt first to cut off the water by grouting, a matter which had been under advisement for some time. As the rock lay in horizontal layers, it was hoped that the water would not



TANK GROUTING MACHINE.

FIG. 2.

follow the shaft down, but could be completely cut off by grouting the porous layers.

A battery of four Canniff tank grouting machines, Fig. 2, was set up at the top with a $2\frac{1}{2}$-in. pipe down the shaft and a 2-in. hose con-

nection at the bottom. At first the grout leaked back into the shaft around the pipes and through seams in the bottom. It was thought that a concrete blanket might be necessary, but the difficulty of placing it would have been great. After several experiments, finely-ground horse manure was injected with the grout. Although there was considerable waste of grout, the manure finally clogged all openings, and the bottom was made tight. In three days 2 900 bags of cement were forced in, and the holes were then blocked. The large quantity of cement indicated the size and extent of the seams. After the grout had set, additional holes were drilled, and struck water under a pressure of 65 lb. at a depth of 14 ft. These holes were then grouted, 60 bags of cement being used.

Experience, up to that time, indicated that the sandstone was very porous and water-bearing, and the High Falls shale lying underneath was believed to be little better. In view of the success of the first grouting, it was decided to put diamond-drill holes down to the top of the Shawangunk grit, and to attempt to grout the intervening strata. The distance from the bottom of the shaft to the grit was 100 ft., and it was decided to bore six holes. At first, 3-in. holes were drilled to a depth of about 5 ft., swedged nipples with gate-valves were driven in tightly, and 2½-in. casing was extended to the top of the shaft where the diamond drills were set up. Three holes were drilled by a Sullivan Badger machine taking a 1-in. core, and three with a Sullivan "C" machine taking a 2-in. core. Water was encountered at a depth of 50 ft. The holes were continued to the grit and 175 bags of cement were forced in under a pressure of 275 lb. The small quantity of cement required was looked on by some as an encouraging sign of better conditions; to others it seemed to cast doubt on the efficiency of the grouting, when considerd in connection with the depth and small diameter of the holes. Both beliefs were shown to have a measure of truth by later developments. The shaft was now making 225 gal. per min., all coming in from the sides.

Sinking was resumed on April 25th, 1909, after a delay of four months, and, in the first 15 ft., numerous seams were passed with a maximum width of 8 in. All were filled with grout, and were tight. Below this point no large seams were encountered, but the seepage from the sides increased until the inflow was 350 gal. per min. at a depth of 280 ft. Trouble with the sinking pumps had become so frequent,

due to the high lift, that it was decided to put in a collecting ring and establish a station at a depth of 265 ft. A sketch of a typical ring is shown on Fig. 3. The pumps on the bottom discharged into the ring, and the water was relayed to the top by two No. 12 Cameron sinkers, which were protected from blasts by a bulkhead of timbers. Sinking was resumed, and when the shaft had reached a depth of 300 ft. and was making 450 gal. per min., a second ring was installed with one No. 12 Cameron sinker pumping to the surface. The second ring intercepted about 100 gal. per min., the upper ring still being used as a station for the two No. 9 Cameron sinkers on the bottom. As the excavation proceeded, a 2-in. seam was encountered, increasing the inflow to 525 gal. per min., and a third ring was added. The water was strongly charged with sulphur; and the H_2S gas, which had been giving more or less trouble, now increased to 9 parts in 10 000. The water caused cuts and abrasions in the flesh to develop into running sores, and the gas, in addition to a general debilitating effect, irritated the eyes greatly, causing temporary blindness. Great difficulty was experienced in getting men, and it was necessary to raise the pay and shorten the hours of work. The bottom was flooded several times, but the water never rose higher than the first ring.

The shaft was now (July, 1910) about 320 ft. deep, and was so full of pumps and their air and discharge lines that it was almost impossible to do any work. A step which should have been taken earlier, but had been postponed by a hope of improvement in the rock, was now decided on, namely, a station chamber in the side of the shaft.

On July 15th the pumps at the upper ring broke down and the shaft was flooded for the sixth and last time. The shaft was unwatered, the six pumps at the bottom were recovered by the invaluable air-lift, and the excavation of the chamber was started on August 1st at a depth of 309 ft. The chamber, as shown on Fig. 2, was 17 by 24 ft. and 10 ft. high, and had a sump 16 by 22 ft. and 5½ ft. high, having a capacity of 14 500 gal. Three 24 by 10 by 20-in. Cameron horizontal condensing pumps, with a combined capacity of 1 050 gal. per min., were installed in the chamber. To relieve the heavy load on the powerhouse, it had been decided to run these pumps by steam, and three 100-h.p. boilers were set up at the top of the shaft. A 4-in. steam line from the boilers to the pumps was wrapped successively with asbestos, felt, and tin, to prevent the condensation of the steam and

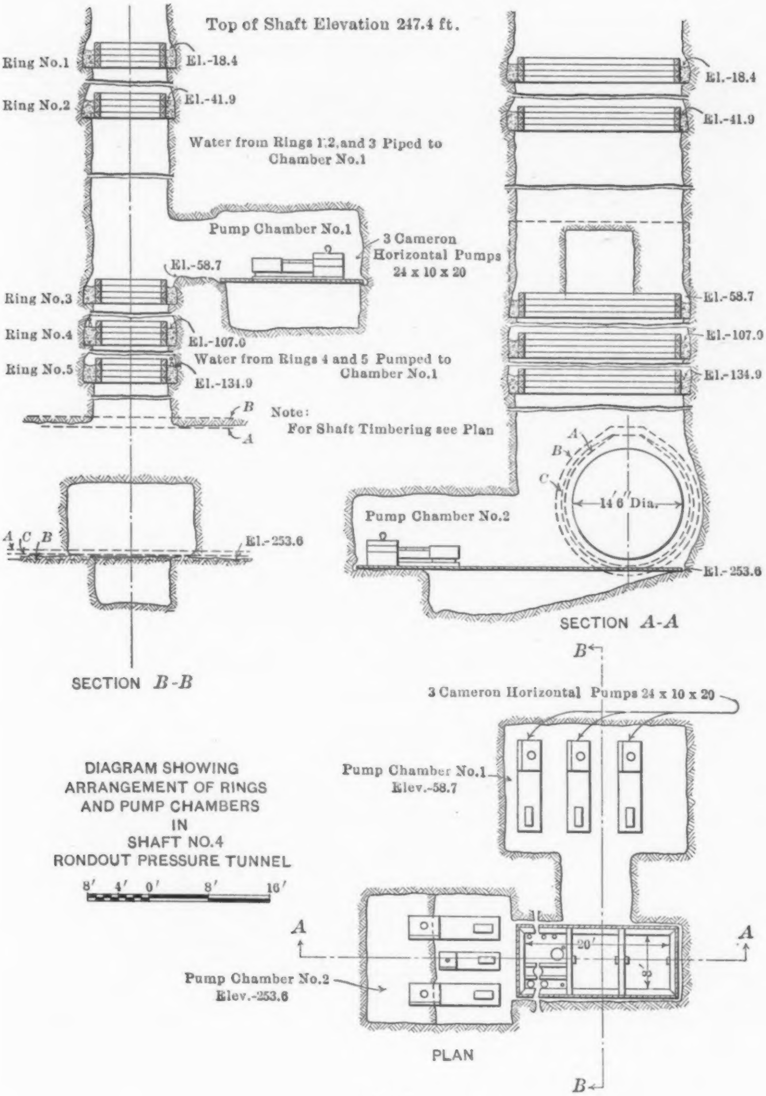

Top of Shaft Elevation 247.4 ft.
Ring No.1
El.-18.4
Ring No.2
El.-41.9
Water from Rings 1,2,and 3 Piped to Chamber No.1
Pump Chamber No.1
3 Cameron Horizontal Pumps 24 x 10 x 20
El.-58.7
Ring No.3
Ring No.4
El.-107.0
Water from Rings 4 and 5 Pumped to Chamber No.1
Ring No.5
El.-134.9
Note: For Shaft Timbering see Plan
El.-253.6
SECTION B-B
El.-18.4
El.-41.9
El.-58.7
El.-107.0
El.-134.9
14′6″ Dia.
Pump Chamber No.2
El.-253.6
SECTION A-A
3 Cameron Horizontal Pumps 24 x 10 x 20
DIAGRAM SHOWING ARRANGEMENT OF RINGS AND PUMP CHAMBERS IN SHAFT NO.4 RONDOUT PRESSURE TUNNEL
8′ 4′ 0′ 8′ 16′
Pump Chamber No.1 Elev.-58.7
20′
8′
Pump Chamber No.2 Elev.-253.6
PLAN

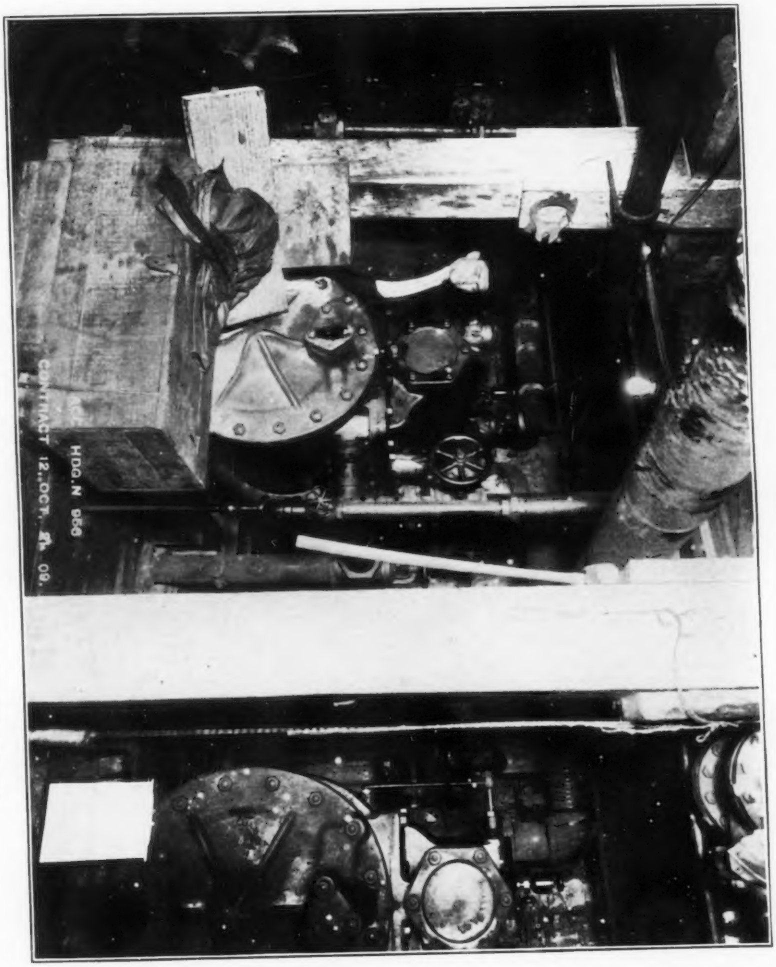
FIG. 3.

fog in the shaft. A water-bearing seam was encountered in the chamber, increasing the inflow to 725 gal. per min., and during the excavation the number of pumps in the shaft increased to ten, of all sizes and styles. By this time the gas had become so bad that it was necessary to ventilate the shaft. Three Sturtevant blowers (two No. 35 and one No. 45) were installed at the top, with 10-in. and 14-in. pipe lines. A mixture of 1 part of chloride of lime and 20 parts of ordinary lime was also sprinkled continuously down the sides of the shaft. After steam was turned on, nearly two weeks were consumed in removing superflous pumps, air-lines, and discharge pipes from the shaft. All water from the rings was either pumped or led by gravity into the sump at the chamber.

Sinking was resumed on September 9th, 1909 (a pilot drill-hole being kept well below the bottom) and all seams were grouted in advance, none of them taking more than 50 bags of cement, until the grit was reached. There was still some seepage from the sides, which sprayed in the shaft and increased the bad effects of the sulphur and gas. Several rings with small horizontal pumps were installed to prevent this. At the contact with the grit an additional flow of 125 gal. per min. came in through the old preliminary drill-hole which had been previously grouted, increasing the total flow to 850 gal. per min. This hole was plugged with a nipple and gate-valve, and registered a pressure of 95 lb. The flow was cut off by forcing in 348 bags of cement under a pressure of 275 lb. All water-bearing seams in the grit were grouted, the maximum quantity of cement used at any point being 100 bags. Sinking was continued until December 31st, 1909, when the shaft reached tunnel grade at a depth of 497 ft., and the headings were turned. The elapsed time from the start of excavation was 18 months, an average monthly progress of 28 ft. Exclusive of delays due to trouble with pumps, the average monthly progress was 45 ft. The highest monthly progress was 80 ft., which was made in the limestone. The total quantity of water pumped during the sinking was 86 181 000 000 ft-gal., and the total quantity of cement used for grouting was 971 bbl.

The general method of excavation was to sink about 65 ft. below the timbers and then place about 50 ft. of timber, the lower dead-logs being about 15 ft. above the bottom of the sump. After large flows of water were encountered, it was found necessary to keep timbers close

PLATE XXVII.
PAPERS, AM. SOC. C. E.
MARCH, 1911.
HOGAN ON
SINKING A WET SHAFT.



PUMP CHAMBER IN SIDE OF SHAFT NO. 4, 309 FT. BELOW THE SURFACE.

to the bottom, for convenience in handling pumps and hanging them and their air and discharge lines. Wire guides were carried down to the bottom of the timbers, and a cross-head or "billy" was used with the bucket. Excavation was made with a center cut or sump and two side or bench cuts, by the continuous method, drilling and mucking proceeding simultaneously, as indicated in Fig. 4. Drilling was started on the cut with four $3\frac{1}{2}$-in. Ingersoll drills mounted on two quarry bars, as soon as the bench muck had been cleared away. The sump was then shot, and drilling was started on one bench while the sump was being mucked. This bench was shot at soon as the sump had been cleaned out, and drilling was then started on the other bench. The average number of holes for a complete advance was 32, and the average advance was about 7 ft. The pumps were hoisted into the timbers while blasting.

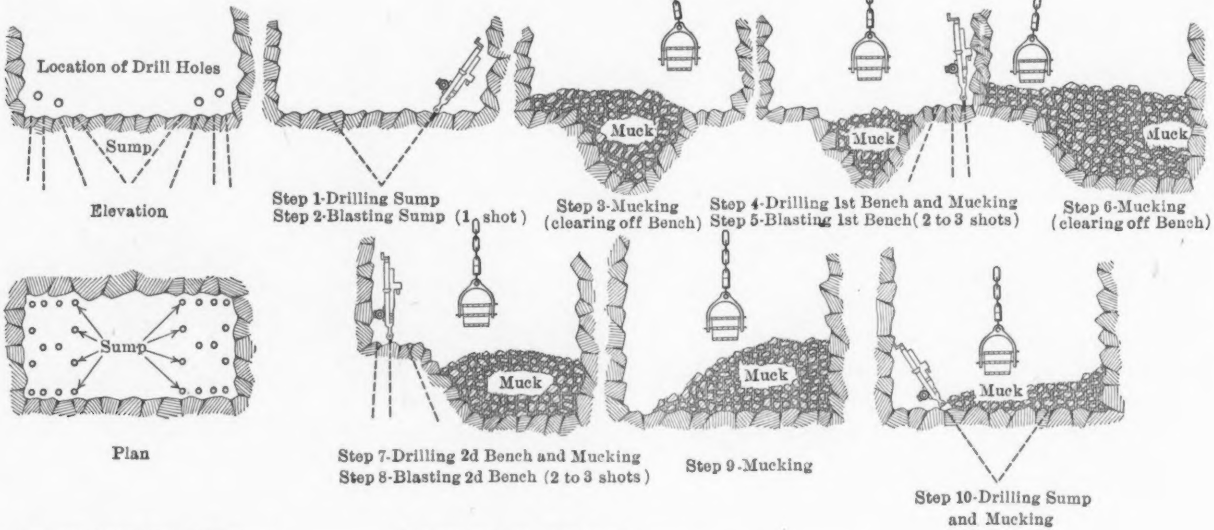
When the inflow became great, the pumps were kept in operation during blasting. The shaft was excavated in separate halves, the pumps being moved to the opposite end of the shaft and protected by a bulkhead. After the station chamber was in operation, the water was kept off the bottom as much as possible by rings, and the former method was resumed. The superintendents and laborers employed at this shaft were mostly trained shaft-sinkers, drawn from the Pennsylvania coal fields.

The methods used in grouting differed somewhat in detail, but in general were as follows: A 2-in. pipe (with gate-valves) was driven into the hole as far as possible. There was great difficulty in getting tight packing around these pipes, and preventing leakage of grout back into the shaft. Wooden wedges, flannel, oakum, oil-well packers, and cement were used freely, but at times it was necessary to resort to the horse manure. A pressure of from 100 to 275 lb. was used, the former being obtained from the main air-line and the latter from a small Westinghouse high-pressure compressor. The grout machines were placed at the top of the shaft and connected with the hole by a $2\frac{1}{2}$-in. pipe. The long pipes clogged somewhat, and cut down the quantity of sand which could be used, but the difficulty of operating the machines at the bottom would have been too great. A battery of from four to six Canniff air mixing tank machines was used, and connected to one pipe. Owing to the simplicity and rapidity of their operation, these machines are admirable for forcing in large quantities of grout,

and kept an almost continuous stream of it passing down the pipe. Between batches, a minimum pressure of 100 lb. was kept on, to hold the grout in place, as the back pressure from the water was at times as high as 95 lb. A very weak mixture was used at first and the quantity of cement was increased gradually; if this continued to flow freely sand was added until the pipe began to clog. All grouting was as nearly continuous as possible, and was uniformly successful (except with the deep diamond-drill holes), at one time cutting off a leakage of more than 400 gal. per min. With the diamond-drill holes, it is believed that, in addition to the length and small diameters of the holes, the partial failure was due to the fact that the chips and dust caused by the drilling clogged the seams. Experience seemed to show that, with the stratified rock (as had been anticipated), the water cut off was not forced into the shaft at a lower level. The grout set very slowly and developed little strength, although the waste grout carried out through the pumps finally set outside.

For any head of more than 200 ft. the sinking pumps gave very poor results, for a number of reasons. The main reliance was placed on four No. 12, 19 by 19 by 12 by 16-in. Camerons, with a 10-in. suction and 8-in. discharge. Each pump weighed about 5 tons and they were so large that they could not be passed by each other in the same compartment. Against high heads they never pumped much more than half their rated capacity, because, when speeded up against the heavy back pressure, the pulsation was so great that it soon tore them loose from their connections. The vibration was so great on a flange-joint discharge line that bolts and flanges broke continually, and it was necessary to replace it by a screw-joint line. One great trouble was the impossibility of making rigid connections with the discharge line. Wire-wound discharge hose, rewound with rope, frequently gave way under the pressure, and the connections between the hose and the pumps were continually failing.

Prior to the installation of the horizontal pumps in the chamber, the air-lift was the one and only reliable part of the unwatering equipment. It was very wasteful of power, but made unnecessary both a large reserve of pumps and the slow and tedious process of staging them down the shaft when it was flooded. It was efficient until the depth of water in the shaft was less than 60% of the total depth of the shaft. When the water was lowered to 40% of the total depth,



Material, Shales and Limestones.
No. Holes Cut 8 x 10 = 80 feet.
Bench, 12 x 8 = 96 feet.
Trimming 8 x 8 = 64 feet.
Total 28 Holes = 240 feet.
Average No. of Holes = 28 for 6⅔ ft. advance.
Average Progress – One advance in 1¼ days.

Yardage per foot = 8.15 cu. yd.
Force: 3 shifts of 2 drillers 2 driller's helpers and 10 muckers.
Dynamite per advance = 120 lb.
Average rate per drill per hour.
Quarry or Shaft bar used.
Cut drilled, shot and mucked North Bench then drilled, shot and mucked.
South Bench then drilled, shot and mucked. Started drilling bench
before mucking of cut was finished.

SKETCH SHOWING VARIOUS STEPS IN CONTINUOUS METHOD OF SHAFT SINKING AS USED AT SHAFTS, RONDOUT SIPHON.

FIG. 4.

the air-lift was usually staged. Sinking pumps were always required to take out the last 10 or 15 ft. of water.

Pumps operated by steam required about half as much boiler-horse-power and half as much coal as for operation by air. When it is considered that there was a central power-plant for delivering air, and that the cost of installation, maintenance, and hauling coal must be considered as an extra, it is probable that the steam plant effected no economy. The principal cause for its installation was an insurance against accident or delay due to any break-down or overloading of the central power-plant. Steam was never used with the sinking pumps, on account of the excessive heat and the fogging of the shaft which would result. The steam horizontal pumps used in the chamber were condensing, thus preventing this difficulty. The air gave considerable trouble by freezing up the pumps, although re-heaters and drips were used.

As a source of difficulty and delay in sinking this shaft, the H_2S gas was second only to the water. It is believed to be due to the decomposition and dissolution of pyrites in the rock, under the heavy pressure. The gas arising from the shaft soon oxidized the paint on a building 100 ft. away, and the discharge water would tarnish silver coins in less than a minute. In addition to their debilitating effect on the men, the gas and water attacked pipes, cables, rubber gaskets, and hose with serious effect.

It is believed that an ultimate economy might have been effected by constructing the chamber earlier, and at a higher level, with perhaps a second chamber in a lower position. The completion of the shaft is due to the successful use of the process of cementation, or grouting. The quantity of water which might have been encountered without grouting is largely a matter of speculation, but it is thought that it would have approached the sum of the amounts encountered at different times. One of the things which lead to this belief is the fact that, while ungrouted inflows near the top of the shaft showed a very slow and steady decrease due to the general lowering of the water-table, the rate was apparently unaffected when new flows of from 300 to 400 gal. per min. were encountered at the bottom. Another is the fact that an inflow of about 2 000 gal. per min. was encountered in the same strata in the tunnel, between Shafts Nos. 3 and 4.

All water pumped from this shaft was paid for under the contract

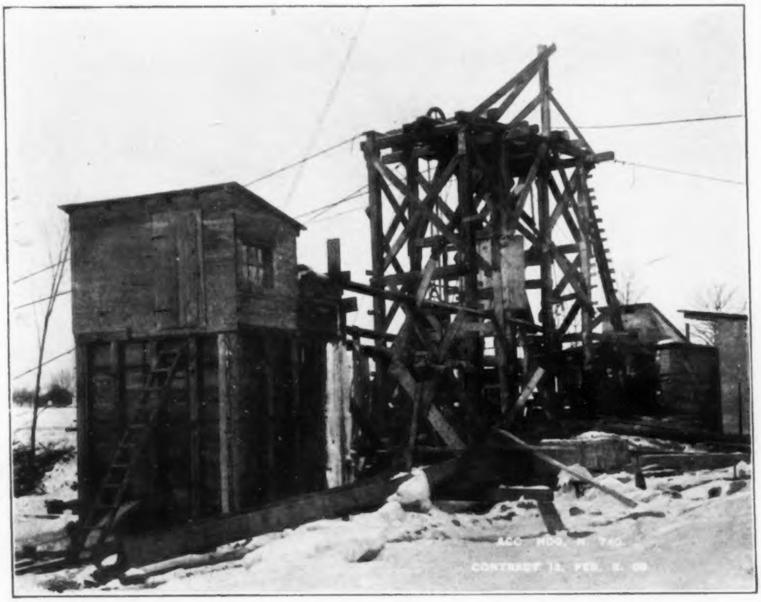


FIG. 1.—HEADFRAME AND WATER-MEASURING DEVICE, AT SHAFT NO. 4.

FIG. 2.—AIR-LIFT, DISCHARGING 1 200 GAL. PER MIN.

at \$0.30 per 1 000 000 ft-gal., and the emergency pumping plant was also paid for under a lump sum, bid price. The water was measured by a 6-in. square orifice, a record of the head being obtained by a Friez automatic water register. The orifice was calibrated and checked from time to time by a box-weir and hook-gauge. The measuring tank and orifice are shown by Fig. 1, Plate XXVIII, and all inflows, unless otherwise stated, are actual registered quantities.

The Rondout Pressure Tunnel is being constructed by contract for the City of New York under the direction of the Board of Water Supply, J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer; Robert Ridgway, M. Am. Soc. C. E., Department Engineer, Northern Aqueduct Department; Lazarus White, Assoc. M. Am. Soc. C. E., Division Engineer, Esopus Division; and Mr. Bertrand H. Wait, Section Engineer, Sections Nos. 5 and 6. The contractors are the T. A. Gillespie Company, Mr. Robert Swan, Vice-President and General Manager, and Mr. R. J. Gillespie, General Superintendent.

The writer wishes to express his thanks to Mr. Wait, who was in actual charge of the work, and collected the data from which this paper was prepared.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion.*

By Messrs. Logan W. Page, W. W. Crosby, Arthur H. Blanchard, Clifford Richardson, Harold Parker, Henry B. Drowne, S. Whinery, J. L. Wickes, H. C. Poore, Charles W. Ross, Prevost Hubbard, Frank J. Eppele, John R. Rablin, R. K. Compton, William H. Connell, and F. C. Pillsbury.

PRELIMINARY INVESTIGATIONS.

Mr. Page.

Logan W. Page, M. Am. Soc. C. E.—There are at the present time many features in American practice which render the satisfactory application of preliminary investigations to highway construction unnecessarily difficult. It is the object of this discussion to point out some of the most prominent of these obstacles and to suggest certain means of remedying them.

The present status of experimental road work in the United States is far from satisfactory. It must be admitted, however, that much progress along experimental lines has been made during the past few years by certain individual investigators and public service bodies, and a general improvement in methods of construction and plans for the investigation of previous work is noticeable throughout the country. The best that can be said, however, is that a start only has been made along the right lines, and that much yet remains to be done.

Since 1907 a great number and variety of experiments with dust palliatives and road binders, in the treatment and construction of roads, have been conducted by American road engineers, but the greater part of this work has been of practical value to the experimenter only. Many valuable experiments have not been published in such form as to assist others engaged in similar work, and the general criticism

* At meetings held January 20th and 21st, 1911. As much of this discussion as possible is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

may be made that in the majority of published reports the data presented are both meager and incomplete. Moreover, the results obtained from these experiments have not been properly correlated or digested. This, of course, may be accounted for, to some extent, by the fact that sufficient time has not yet elapsed to obtain final results, particularly in experimental road construction. Mr. Page.

Without entering into detail, it may be said that at present there is a lamentable lack of knowledge among road engineers, as a class, regarding even the types of dust preventives and road binders now obtainable; and in many cases there is complete ignorance concerning the physical and chemical properties of such materials. This seems to be a rather unfortunate state of affairs when it is realized that such knowledge is essential to their intelligent use.

Without in any way belittling the good work already done, and the earnest endeavors of many engineers, one other criticism may be made in regard to the experimental road work of the past. This refers to the lack of co-operation among experimenters regarding the general lines which have been followed. Thus, in certain instances one may find that a bituminous material used satisfactorily by one experimenter in construction work has been used by another in surface treatment with bad results, as might have been expected. Without attempting to ascertain the cause of failure, one will discredit this particular material while the other will extol it. A third party wishes to surface-treat or construct a road, and, hearing opposite opinions from the first two experimenters, is at a loss to know whether or not to use the material in question. As a rule, without regard to the experience of either, or taking the trouble to find out the cause of the difference of opinion, he decides to make an experiment of his own, and often repeats mistakes which have previously been made.

To remedy this condition of affairs, the following suggestions are offered:

(1) That all engineers interested in modern road problems make a special study of bituminous and other road materials, with regard to their origin, classification, methods of production or manufacture, and their physical and chemical characteristics;

(2) That a systematic examination be made of all dust preventives and road binders used in experimental work, and that complete reports of such examination be included in all published descriptions of experiments;

(3) That a detailed description of the method of application or construction be made in all published reports, and that a statement of all peculiar local conditions in connection with each experiment be included in these reports;

(4) That, whenever practicable, a careful traffic census be taken for all roads which have been surface-treated or constructed with bituminous or other binders;

Mr. Page.

(5) That more attention be paid to comparative experiments carried on under a given set of conditions, where various road binders are used according to one or more methods;

(6) That experiments be conducted with the view of determining the relative economic value of surface treatment and construction with various road binders, especially in connection with the traffic to which the road is subjected.

The first of these recommendations may be passed without further comment. With regard to the second, it may be said that the methods of examination of road binders should be standardized, in order that the results of examination be properly interpreted by all. For a number of reasons, such standardization is a difficult matter to accomplish, but it is to be hoped that certain standards will be generally adopted in the near future, as a result of the work of this Society and the American Society for Testing Materials.

With regard to the construction of experimental bituminous roads, it is suggested that more attention be paid to the economical development of the mixing method, which, in the speaker's opinion, will eventually prove best adapted for the more important highways.

In the speaker's judgment, the importance of taking a traffic census has been greatly underestimated in the United States It is quite evident that the selection of a suitable type of road for any locality should be governed largely by the volume and character of traffic to which the road will be subjected, and yet but little attention is commonly given to this matter. Attention is rarely given to more than the general nature of the traffic; that is, whether it is heavy or light, and whether it consists mainly of motor vehicles or horse-drawn vehicles. What is considered as heavy traffic for one locality may be considered as light for another, and, therefore, it would seem to be most important that a detailed record be made of both the character and the volume of traffic for each individual road of importance.

With regard to comparative experiments made under the same general conditions, there is one point which should be emphasized, namely, that the results of such experiments should be interpreted, not necessarily as showing unquestioned comparative values, but mainly comparative values under a given set of conditions. This leads to a closely related subject which may be mentioned in conclusion, that is, the futility of attempting to formulate standard specifications for bituminous road materials. It has been the speaker's experience that a given binder, used according to a prescribed method, with satisfactory results under a given set of conditions, may prove quite unsatisfactory when used according to the same method, under a different set of conditions. The framing of satisfactory specifications for bituminous binders, therefore, must continue to be, as in the past, a matter of expert judgment.

W. W. Crosby, M. Am. Soc. C. E.—In the work of preliminary investigation it is necessary to consider how the actual performance of the work to be accomplished may be carried out, before a decision as to the method to be adopted may be reached. Mr. Crosby.

Owing to the fact that the speaker has so frequently heard expressed the desire for a panacea or cure-all for road-ills, in the way of a particular method of using any material, and that he feels there is still remaining, in some quarters at least, a mistaken idea of his attitude on these questions, in spite of his previous utterances on this subject, he desires to make a brief statement of his position.

Any tendency to establish a panacea for either method or material is to be regretted, because any such idea is improperly based and impossible of accomplishment, and because each problem should be solved individually.

The speaker believes that even the relative values of the four common methods in use, namely, Palliatives, the Surface Treatment, the Penetration Method, and the Mixing Method, have not been as clearly established from observed and recorded facts as is necessary for a final opinion as to which method should be used, even when all other circumstances in the case may be accurately known. The speaker considers that each method has its limitations, though they may not be sharply defined at present, and that there are many good arguments for each.

Palliatives have their main use on roads of light traffic, already in good condition as to surfacing; where excessive smoothness is undesirable; where there are esthetic objections to other methods; where their convenience of application demands serious consideration; and where annual cost may not be a controlling factor.

Surface treatments are limited to roads with good surfaces; of moderate traffic; where a fair degree of smoothness of surface and the resulting appearances from "glaze" or color may not be objectionable; where the cost under the traffic may not be excessive; and where the convenience of their application is desirable of consideration. It will probably be generally acknowledged that the records as yet are not clear as to the amount of traffic that can be cared for successfully by the surface treatment, or as to its economy under known traffic conditions.

Penetration methods are limited to the resurfacing of old roads or the construction of new ones. Their relative economy is by no means clear as yet, nor is their relative efficiency under known traffic conditions apparent. A great argument in favor of their adoption—in addition to the theoretical ones which have probably been covered too fully in the past to need repetition here—has been the practical one that it was so much easier to induce lay boards, or officials, controlling the work, to "sprinkle some pitch on the stone before applying

Mr. Crosby. the chips," than it was to induce them to invest a portion of their limited funds in the equipment necessary for the mixing method. Undoubtedly, this condition of affairs has also been furthered by the attitude of certain patentees, who have so often made such broad claims, as to the extent of their patents, as to scare many laymen into avoiding any mixing method, even when they were convinced that some such might be desirable, and were ready to make the investment for equipment. Consequently, the penetration method has secured an additional argument, perhaps in some cases unjustly, though the speaker wishes to avoid any lack of clearness as to his opinion. He still firmly believes in his previous arguments as to the correctness of the principles underlying the method, and he submits that as yet no appreciable amount of direct evidence has been recorded to disprove either these theories or the actual value in practice of the method for many cases. The penetration method is probably limited in its actual use to roads with more than a minimum of traffic, but what that minimum is, as yet, is not satisfactorily clear; to roads where extreme firmness and, perhaps, smoothness of surface are not objectionable; where the dark color is not a serious cause for criticism; and where the cost in the long run is not excessive. It may be an especially valuable process for reconstruction.

The mixing method is also limited in application to reconstruction or new construction. It may be said that the first three methods apply to maintenance and the last two to construction, making the third (the penetration method) lap the dividing line. The mixing method is also probably limited in use by the same considerations as in the case of the penetration method, though the difference in first cost of the two methods may result in some variation in the solution of a problem.

The value of a definite form of treatment is largely dependent on the amount and character of the traffic. The present method of taking records is to count the traffic under certain classifications, as one-horse, two-horse, three-horse, etc., for the horse-drawn traffic, and the light and heavy motor cars and motor trucks for the motor-car traffic. In the speaker's experience, a comparison of records taken in this manner is extremely difficult. For instance, on various roads, he has records which have been accumulating for a number of years, and, even when they are all tabulated, it is difficult to compare the results.

If engineers will experiment a little in obtaining such information, and assign a unit to each class of traffic, so that the data can be reduced to units, the comparison of results on different roads will be simple. As an illustration, ridden horses might be given a value of 1, single-horse teams the value 2, two-horse teams the value 4, motor cycles the value 1, runabouts or motor cars in general, except trucks, the value of even double the number of seats in the car, and motor trucks a value of 1 for each 250 lb. of capacity, then the traffic could be reduced to a single unit for purposes of comparison.

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Such a scheme is suggested with the idea of obtaining a method by which results can be compared, and so that engineers can judge of the relative value of the different treatments, methods of construction, and possibly of the different materials used.

To sum up: The selection of a method to be used for a particular case, when arranged diagrammatically, may look as complicated as the pedigree of an Arabian horse traced back even a few generations.

Diagrammatic Arrangement of Controlling Factors Influencing the Selection of the Method to be Adopted.

- Method to be used:
  - Physical Suitability of Results:
    - Esthetics;
    - Traffic;
    - Smoothness, or Firmness;
    - Dustiness, or Cleanliness;
    - etc.
  - First Cost:
    - Equipment;
    - Cost of materials;
    - Cost of skilled labor;
    - Cost of ordinary labor;
    - Workable season;
    - etc.
  - Cost of Maintenance:
    - Repair equipment;
    - Cost of materials;
    - Cost of using materials;
    - Workable season, and interference with repairs;
    - Frequency of repairing;
    - etc.
  - Human, or Personal Elements:
    - Control by laymen, or by engineers;
    - Local or personal prejudices or desires;
    - Knowledge of actual facts by controlling authority;
    - Tendency of controlling authority to accept other facts or theories;
    - etc.

Further divisions of this diagram may be carried out almost indefinitely. That which is shown will serve for the illustration intended.

Mr. Crosby. If the problem seems formidable at present, it should not be forgotten that it is surely capable of simplification. It is the duty of each engineer, who is interested in the development of the science of the Profession, to exert himself courageously for the elimination of certain improper factors, and for the substitution of facts for the unknown factors at present perhaps algebraically represented, as carefully as recorded experience and co-operation will enable him to dictate, so that the "answer" may be sharp and clear.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The primary object of preliminary investigations is to enable an engineer to determine within certain limits the method of construction or maintenance and the kind of road material which are adaptable, from the standpoints of economics and efficiency, to local conditions.

It is hoped that by the publication and dissemination of complete data relative to the construction and maintenance of roads, and by the co-operation of engineers through the medium of Committees appointed by National Societies of Civil Engineers, the limits of adaptability of methods and materials may be defined more clearly each succeeding year.

Preliminary investigations should include a consideration, and, in many cases, a thorough investigation of many of the variable influential factors mentioned in the following discussion.

In the study of the traffic to which the road is subjected, it is essential to ascertain the amount and the character of the different types of horse-drawn vehicle traffic and motor-car traffic during the various seasons of the year; the normal and abnormal speed of various classes of traffic; the direction of traffic; the parts of the road occupied by various kinds of traffic; the nature of horses' shoes and non-skidding devices used; the traffic regulations in force; and the probable change in the character and amount of the traffic, due in part to the improvement of the road in question or of other roads.

Engineers familiar with the problems of modern highway construction will recognize the influence of the foregoing elements on the ultimate design. For example, a traffic census showing intense and heavy commercial traffic evidently calls for a different type of foundation than the combined traffic of the same number of vehicles but of the passenger type. Again: from the standpoint of the preservation of the surface, a traffic of 200 motor cars per day may or may not necessitate the use of a bituminous material in the construction or maintenance of a road, the combined speed and weight of the motor-car traffic being the controlling factor.

Physical local environments and conditions affect the design materially. The following are among those which should be noted: topographical and geological structure and features; condition and

character of cross-roads; effect of the character of the surface of the proposed road on the locality, from the standpoint of esthetics; character of existing surface, foundation, and sub-soil; relation of road to system of roads, from the standpoint of possible diversion of traffic during period of construction; existing and proposed grades and crowns; months available for construction and maintenance work; climatic conditions, covering range of temperature and amount and distribution of rainfall and snowfall. Mr. Blanchard.

The availability of the various materials which might be used in construction and maintenance has not, in many cases, received the requisite amount of attention. The investigations along this line should cover a thorough study of the cost, delivered on the site, and the physical and mineralogical character of all rocks, sands, gravels, etc., which might be used on the road. The source of supply of palliatives and binders should likewise receive attention; and the effect of specifications covering the various physical and chemical properties on the ultimate cost of construction or maintenance is worthy of consideration. In some cases, the method of delivery of bituminous materials has affected the cost and the use of certain methods; hence the location of the road, relative to railroads or storage plants, should be ascertained. The character of available labor, the wages paid, and the method under which the work can be accomplished, may also affect the design.

The subject of plant equipment for the construction and maintenance of bituminous surfaces and bituminous pavements has not yet received the consideration warranted by the importance of the subject. It is obvious that, as a result of thorough preliminary investigations, it will be possible, within reasonable limits, to adopt various types of construction and maintenance for a system of highways, in which case it is likewise self-evident that the acquirement of plant equipment will reduce materially the cost of both construction and maintenance under certain conditions.

Clifford Richardson, M. Am. Soc. C. E. (by letter).—The writer is not a great believer in preliminary investigations at the present time, for several reasons, and is inclined to think that they will result in confusion. The problem of the construction of roads which will meet the conditions imposed by modern motor traffic should be dealt with cautiously. In the United States, at least, attempts to solve it have extended over such a short period of time, that as yet no data are available which offer a basis for investigation. The situation is still purely experimental. What engineers are looking for is success. When this is attained, the manner in which it has been reached can be investigated. Mr. Richardson.

While theories may be of some little value, it is only from expe-

Mr. Richardson. rience and service tests that definite conclusions can be safely drawn. Analogies, however, may aid in directing our efforts. Highway engineers, up to the present time, have failed to benefit to any considerable extent from the experience gained in laying bituminous pavements.

It does not seem to be wise at this early stage of the development of bituminous highway construction to draw any conclusions, except those based on analogies, or to make any recommendations, the final results of which can be imperfectly foreseen, for, undoubtedly, they will have to be changed or abandoned in the future. A new industry may be restrained seriously in its development by attempting to prescribe methods of construction which are not based on successful results as demonstrated by service tests. To avoid falling into error, one must await such results and then determine the manner in which these successful results have been obtained, in order to be able to duplicate them. Efforts in any other direction will be in vain. As a parallel, attention may be called to the long period of time which elapsed before it was possible to place the sheet-asphalt paving industry on a rational basis. This was finally accomplished after there were available for study a number of asphalt surfaces which had given satisfactory results for 10 to 15 years or more, under traffic of various kinds. As yet, no bituminous highway surfaces are of sufficient age to furnish the necessary data for a similar study, except in the case of bituminous concrete. The prospect of eventual success of bituminous construction on highways, however, is highly encouraging; that is, there seems to be little question but that some of the present forms of construction, or modifications of them, where tried materials are used, will prove to be a vast improvement over the water-bound broken-stone road, both from the point of view of resistance to the destructive effect of motor travel, and of economy and unit cost per ton-mile of haul.

The data, in connection with any form of construction, which should be accumulated at the present time are: accurate records of first cost; of the cost of maintenance during the life of the surface; the traffic which it carries, as revealed by traffic censuses at intervals, and the environment under which this traffic is carried, in order that the actual cost of carrying it for the life of the surface may be accurately determined. With such data available, the economy of any form of construction may be compared with data of the same description for any other form. It does not seem possible that any definite conclusions of value can be arrived at, at present, or until such data as have been mentioned are assembled. When a successful form of bituminous construction has been demonstrated, investigation will be of value to determine how this can be repeated, but, until this has been achieved, the writer believes that investigations will be of little value.

RELATIVE VALUE OF THREE METHODS OF CARRYING ON WORK:

(*a*) That in which both labor and material are furnished by the contractor.

(*b*) That in which the material is supplied by the party of the first part, and the labor by the contractor.

(*c*) That in which both the labor and material are supplied by the party of the first part.

HAROLD PARKER, M. AM. SOC. C. E.—In opening this discussion, the speaker will state the reasons why, under the conditions which he will mention, the affirmative position should be taken on each of the three divisions under discussion. Mr. Parker.

It must be clear, to any engineer in charge of construction of any sort, that the various conditions under which the work is to be done should control his action in asking for proposals from contractors, and, depending on such conditions, one of the three methods would from necessity have to be adopted.

If it is desired that the cost of the work shall not exceed a fixed sum, and that the estimate previously made by the engineer is to be corroborated, the method defined under "*a*" would be preferable. This would occur in many instances where public work is involved and the cost of its execution is limited by estimates previously prepared. In such cases there does not appear to be any question as to the wisdom of having the contractor furnish both the labor and materials.

In the practice of all constructing engineers, the second method, that is, "materials supplied by the party of the first part, and labor by the contractor," must frequently be found to be the best, if not the only one, by which the work can be done economically and satisfactorily. It has the advantage that the engineer, desiring to use in the work certain types or brands of materials, the cost of which he knows, can be perfectly sure of the final cost before the work is begun. In this case all contractors who bid are placed on complete equality, and, therefore, as an intelligent and comprehensive offer can be made by each, an intelligent comparison can be made.

In the construction of concrete bridges, culverts, or walls, this may be by all means the best way to do the work, because it very often occurs that the engineer desires certain kinds of rods, placed as he plans them, or certain brands of cement or other material, which he wishes particularly to have embodied in the work; and, during the work, or at its conclusion, this method would prevent the question arising as to the use of other brands of material which might justly be claimed to be of equal quality, thus creating controversy between the parties to the contract. At any rate, in cases where it is particularly desired to use materials of a fixed type or character, and where there may be doubt in the minds of experts generally as to which is the best, thereby

Mr. Parker. producing uncertainty in the interpretation of the specifications, there can be no question, if the materials are supplied by the party of the first part, and the labor alone by the contractor. This is so true that the Highway Commission of Massachusetts has for many years followed this practice in the construction of its concrete structures, and the reasons for this appear to be good. At the beginning of each year, the Commission receives bids for cement, to meet certain specifications. This cement is tested at different times, in order to see that it comes up to the requirements. It is furnished to the contractors as it is needed, and their responsibility for its quality is not a part of the contract. In this way, uniformity of material and satisfaction are ensured. The contractor is also supplied with rods, of dimensions determined by the engineer, and these are placed as he directs. The contractor in this case furnishes only the labor and the gravel or broken stone.

Under the third method, where both labor and materials are supplied by the party of the first part, the instances where contracts are let can be but few, if, indeed, it may be said that a contract can be entered into where neither material nor labor are furnished. This method, which, from the nature of things, can involve no agreement between parties, would be preferable in the performance of work of a character which it is impossible or impracticable to define in specifications with sufficient clearness or precision to enable one to make an accurate estimate of the cost. In the performance of such work it is also preferable because special skill and knowledge are necessary, and these have not been acquired by contractors, or, at any rate, by a sufficient number of contractors to secure fair, competitive bids.

From the speaker's experience in Massachusetts and elsewhere, he believes that work under contract should be done under either method *"a"* or method *"b."* If method *"c"* is contemplated at all, it should be when the conditions are such that reasonable bids cannot be obtained, on acccount of the inexperience of contractors, or for other reasons. Therefore, method *"c"* only contemplates a comparison between contract and day labor.

In order to get economical work by day labor, it is necessary that the party of the first part shall have a constant force of laborers and foremen trained in the particular kind of work he has in mind, and constantly employed on work of the same type or class.

It may be, and, in fact, is found, that in the maintenance of certain roads, where the conditions under which a contract could be made are uncertain and involve unknown costs which will not permit the contractor to make a reasonable bid, method *"c"* is preferable.

In maintaining roads in Massachusetts, especially where oil and tar are used, it has been found that the work can be done more economically by the third method than by either of the others; and it is better done, because the maintenance gangs are well organized,

and are under trained foremen, who have learned all they know under the direction of the engineers of the Commission, and, from their practice under very careful supervision, have become expert. Consequently, the work is better done and at a less cost than it could be by letting it to any contractor. Mr. Parker.

It seems to the speaker, therefore, that, where the conditions warrant, any one of the three propositions might be adopted by the party of the first part, but it must be for that party to determine, after most careful consideration, which of the three is best suited to the work in view.

In general, the speaker maintains that, wherever possible, method *"a"* (the contractor furnishing all labor and material) is preferable for all construction, except where reinforced concrete constitutes a large proportion of the work.

Method *"b,"* in which the contractor supplies the labor and the party of the first part the material, or a certain proportion of it, is preferable where the largest portion of the work is of reinforced concrete, or is of an experimental nature, or where a bituminous material of a specified kind is used.

Method *"c,"* in which the party of the first part furnishes both the labor and material, is preferable in cases where the amount and kind of work are difficult to determine, and where it is of such a nature that the contractors cannot make intelligent estimates of the cost.

Henry B. Drowne, Assoc. M. Am. Soc. C. E.—This discussion pertains to the relative efficiency of three methods of carrying on bituminous road construction. For several reasons, the arrangement between the contracting parties is not as satisfactory under the first method as under either of the others. The majority of road contractors are inexperienced in bituminous construction, and, therefore, the bids received are generally out of proportion to the actual cost of the work, and in most cases allow the contractor an unreasonable profit. In doing work by this method it is necessary to have the specifications covering the bituminous material ready at the time the contracts are advertised, or a list of the different brands of material on the market which will be allowed in the work must be given to the contractor. If the contractor selects the material from the approved list, the cost of all rejected material will have to be borne by the party of the first part, and, furthermore, as bituminous materials manufactured to-day and given the same trade name vary so much in quality, the party of the first part would be taking a great risk of not obtaining good results in so far as they may be affected by the material. On the other hand, if the bituminous material is described in the specifications, it may be found, during the progress of the work, that, although it has fulfilled the conditions imposed, it is not at all satisfactory, and a change may be desirable. These changes may affect the characteristics of the Mr. Drowne.

Mr. Drowne. material, so that the cost of manufacturing such a product may be considerably less than for that first specified. Although the contractor is generally willing to agree to any changes in specifications which increase the cost of the work, provided due allowance is made for the same by the party of the first part, it is unusual for him to agree to any reduction in his contract price, although it can be clearly proven that any proposed change in specifications would reduce the cost of the bituminous material which he is obliged to purchase. The same remarks are true in regard to the specifications as to the method of doing the work. As far as the rate of progress of the work is concerned, the speaker feels that, although more actual yardage may be obtained per day when the work is let by contract, the quality is apt to be sacrificed, as the contractor is much more interested in the quantity than in the quality. Extreme care in doing the work is one of the essential elements of successful bituminous roads.

The only advantage that the speaker can see in performing the work entirely by contract is the saving in office labor that would ensue if the party of the first part furnished the bituminous material and had the work done on labor account.

When all road contractors have become familiar with bituminous road construction, and the engineers are so well informed that they can specify a certain bituminous material and method of construction and know that the resulting road should be good for a definite number of years, then the work could be let out by contract, and fair prices would probably be obtained. If the party of the first part employed intelligent and competent inspectors, careful work on the part of the contractor would be assured. The use of a guaranty clause in the contract, however, is rather a doubtful expedient for securing good work. A poor piece of work can sometimes be kept in repair for the number of years stipulated in the guaranty at a small cost, but at the end of this period the road may need such extensive repairs that a renewal would be necessary. Moreover, the cost of the maintenance for the years covered by the guaranty has been paid by the party of the first part, because the contractor has made allowances for this in his bid.

The speaker has had considerable experience with the second method, in which the bituminous material is supplied by the party of the first part and the labor by the contractor. The contracts were advertised as ordinary macadam, and bids were received for stone in place and grading. If it was decided after signing the contract that the work was to be of bituminous construction, the contractor was informed, and a separate agreement was drawn up and signed by both parties. This agreement covered the prices for labor furnished, all allowances for equipment, etc., necessary for the bituminous construction, and any deductions that were to be made from the contract price for work to be done on labor account and already included in the contract.

Although in no case did any contractor refuse to sign the agreement, some were reluctant to agree to any deductions. The party of the first part paid the contractor the amount of the labor bills for the bituminous work, minus whatever deductions were agreed on, plus a 15% allowance on the remainder, and plus the allowances for equipment. Mr. Drowne.

Different methods of construction were tried, and on one road both the method of construction and the material were changed several times, with no objection on the part of the contractor. It seems to the speaker that reserving the right to change anything in connection with the bituminous work without having any trouble with the contractor is a great advantage. The party of the first part, in purchasing the bituminous material direct, saves any profit which might have been made by the contractor if he had purchased it, and by paying the actual cost of the work plus a 15% allowance, it can be expected that the cost will be as low as possible.

The rate of progress, by doing the work in this way, is probably not as great as if done by contract. The labor, however, is not the large item in the total cost of bituminous construction, and so a fair rate of progress with the work done in a thorough manner should be satisfactory. A comparison of this method with that of doing the bituminous work by contract came under the speaker's observation two years ago. In the first instance, the contractor had built a bituminous pavement by contract for 22 cents per sq. yd. over and above the cost of ordinary macadam. In the second instance the conditions were exactly the same as in the first, in so far as the parties to the contract, location of road, method of construction, and materials were concerned, but the bituminous material was furnished by the party of the first part, and the contractor did the bituminous work on labor account. The cost of the work in the second instance was 12 cents per sq. yd. over and above the cost of ordinary macadam. The price for the ordinary macadam was determined by contract. An allowance was paid to the contractor for the use of the extra equipment for the bituminous work (consisting of two kettles, fuel, mixing boards, dippers, and long-handled shovels), and also a 15% profit on the labor bill. For this work there were no overhead charges except on the extra equipment involved. The allowance made for this by the party of the first part more than paid the actual depreciation, and made the 15% on the labor practically clear profit. The price of 12 cents includes everything mentioned above, and is, therefore, the true cost which can fairly be compared with the cost of 22 cents.

Some contractors object to this method, mainly because the work may be slower than if ordinary macadam were built. For example, on a pleasant day immediately following a rain it is possible to do ordinary macadam road work, but it is not possible to do any bituminous construction. The bituminous gang has to be paid at such

Mr. Drowne.

times in order to keep it, and if the men work, it is oftentimes at a disadvantage. With the better class of contractors, this method has worked well, and the time for completion of construction has not been much longer than if ordinary macadam had been built.

It is the speaker's belief that this method is the best of the three, provided that, if a bituminous road is to be built, it is advertised as such, and that some clause is added to the extra work clause in the general specifications to include the bituminous construction as extra work, subject to agreement between the two parties. If such a course is followed, the method is fair to the contractor, and should result in good work at a low figure.

The advisability of using the third method, the party of the first part furnishing all material and labor, depends to a great extent on conditions. Excellent results ought to be obtained by having the work done by a gang of men trained specially for the purpose. As the party of the first part has also an unhampered choice of materials, there is no reason why those best suited to the road could not be obtained. There is no doubt that maintenance work and the application of a bituminous surface to roads already constructed could be accomplished economically by this method, as these particular branches of the work do not require either large gangs or much equipment, and the rapidity with which the work can be done allows a few gangs of trained men to cover a large territory. Any large amount of new construction in one season could not be carried out by the smaller municipalities and towns, on account of the large equipment required to do the work. In places where an efficient public works department is maintained and in other places where sufficient work is contemplated to warrant an outlay for the necessary equipment, this method might be used to advantage.

If the bituminous work was to be done by the mixing method, the speaker believes that it could be done economically by establishing bituminous mixing plants at points central to the different jobs, and by shipping the mixed stone ready to be put into place. There are several machines now on the market which can do a great deal of work provided they are properly set up and operated and the aggregate is furnished in quantities commensurate with their capacity.

The success of the third method, if conditions warrant its use, depends to a large extent on the efficiency of the corps of laborers and foremen employed by the party of the first part. Politics creep in on most public work, so that oftentimes many incompetent men are employed, and, although civil service remedies this evil, it is almost impossible to eliminate it entirely. Many instances might be cited where the labor cost in doing the work by this method is as cheap or cheaper than if the work was done by contract, and still, as a general rule, the opposite is true.

THE USE OF WATER, CALCIUM CHLORIDE, LIGHT OILS, ETC., AS DUST PALLIATIVES.

S. Whinery, M. Am. Soc. C. E.—In the discussion of this topic the element of road preservation is excluded, except in so far as the use of the materials described may incidentally affect that problem. The field for discussion, therefore, is a narrow one, confined principally to the relative merits and economy of the various materials for allaying dust. Mr. Whinery.

The best known and longest used method of making and keeping streets and roads dustless is by sprinkling with water. At the same time it may be said to be the least developed process, as far as scientific study and application are concerned. Accurate and extended observations on the effect of sprinkling roads and streets, the cost of the process, the quantity of water required, the best and most efficient method of applying the water, and other pertinent matters have either not been made or the results have not been published. If any one doubts this, let him attempt to obtain, from the scanty literature relating to the subject, reliable and consistent data on any of these points.

Municipal engineers seem to have always looked on street sprinkling as an every-day, matter-of-course operation, not worth serious attention and study, and not susceptible of much improvement in efficiency and economy. It is safe to say that no other municipal function or process receives so little intelligent attention and supervision.

When the sprinkling season arrives, the usual programme is to equip a more or less unintelligent and uninterested driver with a sprinkling wagon and a hydrant wrench and turn him loose to work his sweet will on the street to which he is assigned. His autocratic reign is only interfered with, and then but temporarily, when outraged residents have the temerity to complain to the authorities supposed to be in control.

It is not surprising, therefore, that municipal engineers and the public have a vague feeling that street sprinkling as a method of dust suppression has proved more or less unsatisfactory and inefficient; but, before condemning it and advocating a substitute, it should have a fair trial under the same conditions of intelligent use as its newer competitors.

The cost of efficient street sprinkling is difficult to determine satisfactorily from the data available. Where referred to at all in municipal reports, the necessary conditions to enable one to deduce reliable figures of unit cost are usually wanting. These reports indicate that the cost, exclusive of water, ranges from less than 1 cent to 3½ cents per sq. yd. for the watering season, which usually extends from May 1st to October 15th. The number of square yards sprinkled once is seldom

Mr. Whinery. given, and as the number of times sprinkled per day or per season varies greatly in different cities, and on different streets in the same city, and as there are other varying conditions to be taken into account, it is not possible to get at the basic unit cost from the available records. It is practicable, of course, to estimate the theoretical cost very closely, but such estimates are less satisfactory than actual data from experience.

The statistics available seem to indicate that, exclusive of the cost of the water, the average cost per square yard per season is about 2 cents. The quantity of water reported as used is as variable as the cost of applying it. The figures vary from 20 to 70 gal. per sq. yd. per season. Probably 45 gal. is a fair average, and, at $90 per 1 000 000 gal., the cost for water per square yard per season would be about $\frac{4}{10}$ cent, making the average total cost less than $2\frac{1}{2}$ cents per sq. yd. per season. In Boston, where the accounts are now kept with care, the quantity of water used in 1909 was less than 25 gal., and the total cost 2.1 cents per sq. yd. per season. In the suburban City of East Orange, for the same year, these figures were, respectively, 58 gal. and 2.82 cents, including the cost of water.

If it be asserted that street sprinkling as it is generally conducted has not proved satisfactory, it may be replied that it has seldom, if ever, had a fair trial. If the surface of the street were first cleaned, as is the practice before oiling, the objection that water makes the street muddy and sloppy would be removed. If the specifications required that the street surface be kept constantly moist, regardless of whether one daily sprinkling or five were necessary, there would be no trouble from dust. Where the road material is thus kept properly moistened, the wear of the pavement from travel is not greater than where light oil is used, and it is conceded that, as long as the road material is kept well moistened, the peculiar form of disintegration called ravelling does not occur.

As to cost, there is good reason to believe that with careful, intelligent, and economical management, including the restriction of the quantity of water used to that actually necessary, it would not much exceed the average cost of the work as now unsatisfactorily done. Ordinarily, at least, it should not exceed 3 cents per sq. yd. per season, and this figure may be safely used for comparison with other methods.

Calcium chloride has not been used very extensively in the United States, and, if the speaker is correctly informed, this process is now practiced on a large scale in only one city—Boston, Mass.—where its use has been developed with intelligence, through several years. There, the speaker believes, it is considered fairly successful and satisfactory. According to the Report of the Superintendent of Streets for the season of 1909, 408 511 sq. yd. of roads and streets were cared for during the season by this process, at an average cost of about

Mr. Whinery.

$4\frac{1}{7}$ cents per sq. yd., or nearly double the average cost of the total sprinkling in the city. This comparison, however, may not be entirely just, because the treatment was used in only two districts—West Roxbury and Dorchester—and, as the report gives the separate cost of sprinkling in these districts, it will probably be fairer to compare the work in them alone, where the average cost of sprinkling was somewhat less than $2\frac{1}{2}$ cents per sq. yd. for the season. The areas treated by the "calcide" process and by water were approximately the same.

It is reported that the average length of time between treatments was 27 days. Experience in other places seems to indicate that more frequent treatments are necessary, in order to keep the streets satisfactorily free from dust. To some extent, the permissible period between treatments depends on the humidity of the air, and, therefore, one would expect that in seacoast cities a single treatment would remain effective longer than in inland cities. Where a long period elapses between treatments, it is natural to expect that there are many days when the humidity is very low, and the streets become unpleasantly dusty. Observations show this to be the fact. During days of such low humidity a satisfactory condition of freedom from dust can be secured only by resorting to sprinkling with water, in order to supply the deliquescent salt with the necessary moisture.

On the whole, there seems to be nothing in the experience of American cities to warrant a preference for this process, since it appears to be less effective and more expensive than sprinkling.

The use of mineral oils for allaying dust is now quite popular with city and road officials, and with the public. The speaker's remarks will be confined to the consideration of the lighter mineral oils, applied primarily for suppressing dust. That the proper application of light oils will suppress dust effectually for some length of time is undisputed; but, is the use of this material more satisfactory, effective, or economical than sprinkling with water? To answer this question positively requires more complete and accurate data than are yet available. Judging from such facts as are now obtainable, however, the speaker believes that it may be safely answered in the negative. To discuss the matter with any degree of fullness would require more time than is now allowed.

That the oil, for a few days after its application, is a nuisance on the streets and results in no little damage to personal property will be conceded. The length of time for which a single application will be effective depends, of course, on various conditions, which, unfortunately, are not often given clearly in the records.

In these reports the impression is often conveyed that the oil treatment is effective for the whole season, while, in fact, it is necessary to resort to the use of water for a part of the time in order to lay the dust. In practice it is often found advisable not to apply

Mr. Whinery. the oil until the weather becomes settled and the street well dried out, and, where only one application is made, the street frequently becomes very dusty before the end of the season. In such cases sprinkling is often resorted to for a part of the season. For instance, a suburban street, on which the writer lives, was treated with a coat of oil on May 7th. By July 1st the street had become very dusty, and sprinkling with water was resumed until July 20th; then a second coat of oil was applied, which kept the street in good condition until about October 1st. The total cost of sprinkling with water on this street for the season was 1.13 cents per sq. yd. In many such cases the fact that sprinkling had to be used for a part of the season is not mentioned.

In so far as the oiling tends to make the surface of the street impervious to water, it promotes ravelling, due to the absence of moisture. The street referred to above was thus affected toward the end of the season, as there was little rain, whereas, during the previous season, which was also dry, the street was sprinkled with water only, and no ravelling appeared. During the previous year the contract cost of sprinkling for the season was less than $400, including the cost of water, while the cost of oiling in the present year was about $600, exclusive of the supplemental sprinkling. The oil was not more effective than water in preventing dust; it was clearly deleterious to the macadam, and was 50% more expensive. It is to be remarked, also, that in this case the cost of the oiling was unusually low. If the extra cost of oiling had been incurred in securing a superior job of sprinkling with water, there can be no reasonable doubt that the condition of the street would have been decidedly more satisfactory during the season.

One objection to oiled streets (observed in the case previously mentioned) seems to have attracted little attention, but, nevertheless, it is worth noting. During the period when the effectiveness of the oil is declining and the street is becoming semi-dusty, high winds pick up the more or less oily particles of dust and deposit them in the houses or on the clothes of passers-by. This oily dust has a great capacity for soiling articles on which it settles, and it is difficult to remove it by ordinary means.

Other methods of allaying dust on streets and roads have been proposed and tried, but, as they have not proved acceptable or economical, they need not be referred to in a brief discussion like this.

Summing up the conclusions, it may be asserted confidently that no method of suppressing dust, thus far discovered, is more effective and economical than sprinkling with water. Why not, therefore, drop the fads which are being exploited before the public, and turn attention to developing and perfecting this old method, applying to it the same study, care, and efficient management that are devoted to other lines of municipal work.

The speaker believes that there is no practical or mechanical difficulty in developing a power sprinkler which will have double the water and speed capacity of the old sprinkling wagon, and which, at one passage along a street 50 ft. wide, will sprinkle the whole surface effectively from curb to curb. Proper specifications and rigid supervision would accomplish a revolution in the work, both from the standpoints of efficiency and economy. Mr. Whinery.

In a great majority of cases, the common objection that sprinkling the country highways is impracticable because of the frequent absence of a water supply, may be overcome by temporarily installing suitable power pumps at streams and ponds. Where this is not practicable, high-speed power sprinklers would make it possible to obtain the water from comparatively distant sources without increasing the cost of the work materially. With proper watering, the durability of country roads during dry seasons would be increased so much that the sprinkling might almost result in a profit.

What seems to be most needed, for a satisfactory consideration of this subject of dust suppression, is the application of economic and business principles to the several methods, and the collection and dissemination of full and accurate data relating to each. Until such principles are applied, and such data are collected, the indefinite and unprofitable discussion, so common at the present time, will continue.

J. L. Wickes, Assoc. M. Am. Soc. C. E.—In Baltimore, Md., after experimenting for several years with both light and heavy oils, it was finally decided that, from the standpoint alone of a dust-layer, an emulsion was efficient and much more economical than any of the heavy oils. Accordingly, during the season of 1910, nothing else was used. Mr. Wickes.

The emulsion was prepared under the specifications of the Street Cleaning Department. The first three carloads of oil ordered in the spring were shipped in iron drums, each containing about 110 gal. These drums were distributed by teams to the various macadam streets. All subsequent quantities of oil were delivered in tank cars, and the drums were used for unloading the cars and for storage purposes. There was quite an appreciable reduction in the freight charges by this method, and it is probable that considerable saving was effected.

The method of removing the oil from the tank cars and filling the drums was as follows: The yard where the filling was done had a railroad siding which was high enough to allow the oil to flow by gravity through a 4-in. pipe attached to the bottom of the car and empty into the drums. What is known as a tank-car coupling was used on the bottom, and at the coupling the size of the outlet pipe was reduced from 5 to 4 in. Below this there was a short 4-in. nipple, with a 4-in. stop, and a 4-in. bolted flange union. This stop and union were used when the railroad company had to shift the cars, so that the flow of oil could be stopped and the car quickly disconnected. From this union, a

Mr. Wickes. 4-in. wrought-iron pipe was laid for a distance of 30 ft., with a 6-in. fall so that the oil could flow freely. At the end of the 4-in. pipe there was a T or cross, and from this there were two 2-in. wrought-iron pipes with stop-cocks, so that two drums could be filled at the same time.

From time to time some difficulty was caused by lumps of asphalt clogging the pipe and impeding the flow of oil; this, however, occurred only in chilly weather. At such times the asphalt base in the oil congealed and formed in large lumps, and it was then found necessary to stir the oil in the car, in order to make it flow freely into the drums. The outlets to these two 2-in. pipes were high enough to permit the drums to be rolled under them, and a large funnel was sometimes used in filling.

There was great difficulty at times in removing the oil from the drums with the siphons, because they became clogged, and there was not sufficient pressure on the plugs to lift the oil from the drums. This was noticed particularly at the low-pressure plugs; at the high-pressure plugs there was no difficulty.

The first application was made with 20% of oil to 80% of water. In the second and subsequent applications the percentages of oil varied from 10 to 5, according to the weather conditions and the amount of traffic on the road. It was found necessary to treat some of the heavy-traffic roads once a week; some of the lighter traveled thoroughfares were treated only once in 2 or 3 weeks, which was found to be sufficient. The length of road treated aggregated about 20 miles, with an area of 441 263 sq. yd., at a total cost of $9 085.63; of which amount, $5 583.92 was for oil and $3 501.71 for distribution and application. These two amounts include all the expenditure for this work, with the exception of the water used and supervision. The entire supervision, however, was done by the speaker in the course of the discharge of other duties, so that no additional expense was incurred on this account. Hence the cost per square yard was a trifle more than 2 cents ($0.0206), which is less than one-half the cost of applying any of the oils used in this work.

The fact that not a single complaint was received relative to dust on any of the roads treated as above described is good and sufficient evidence that the work was done well and efficiently.

For the benefit of those who are contemplating the use of an emulsion instead of straight oil for the treatment of roads, a synopsis of the specifications and report of Messrs. Penniman and Browne, the chemists who made the tests of the material, are given:

SPECIFICATIONS FOR THE COMPOUND.

The compound must mix with water to form an emulsion so perfect that it can be applied to roads by an ordinary sprinkling wagon without noticeable separation or clogging of the jets. It must be free from

objectionable odor, and must consist of at least 75% of asphaltic oil or base. Mr. Wickes.

This asphaltic base must have the following characteristics:

(1) When 20 grammes of the material are heated for 7 hours in a dish, $2\frac{1}{2}$ in. in diameter, to 205° cent., the weight of the residue shall be at least 65 per cent.

(2) When 20 grammes are heated for 21 hours in a flat-bottomed dish, $2\frac{1}{2}$ in. in diameter, at a temperature of 105° cent., the loss shall not exceed 5 per cent.

(3) The percentage of "free carbon" shall not exceed 3 per cent.

(4) The percentage of paraffin shall not exceed $1\frac{1}{2}$ per cent.

(5) The consistency of the base shall be such that 50 cu. cm. shall flow through an Engler instrument in not less than 185 sec. nor more than 300 sec., at 100° cent.

(6) The consistency shall be such that a Lunge tar tester will sink to the mark, 1.4, in not less than 30 sec. nor more than 45 sec. at 25° cent.

Specifications for the Asphaltic Oil.

These are the same as indicated above, except that the base itself is specified and the following clause is added:

(7) It must form a proper emulsion, using the method of the Street Cleaning Department of Baltimore City.

Method of Making Emulsion Oil.

Dissolve 5% of common resin soap in 30% of water and bring to a temperature of 50° cent. Put this soap solution in a churn and add 65% of oil at the same temperature, churn for 20 min. and run into barrels or tanks through a cooler which reduces its temperature to 21° cent. or less. A portion of the compound prepared in this way must remain unchanged for 48 hours at a temperature of 38° cent. When 20% of the compound is mixed with 80% of water at 32° cent. the emulsion formed must remain without separation for 3 hours.

Theory of the Foregoing Specifications.

The base is specified so that it is sufficiently liquid to be broken up by the paddles of the churn at 50° cent., but, on being cooled to 25° cent., the small particles do not run together, being covered with a film of soapy water. The temperature of 38° cent. is also inserted as an additional practical safeguard to prevent the concentrated emulsion from breaking down before use. The dilution test is introduced for the same reason. The manufacturer of the base is thus compelled to test his product according to the foregoing method of procedure, and shipments are rejected if they are not suitable.

The churning arrangement is not very important, provided the action is that of beating and not merely stirring. A butter churn

Mr. Wickes. is efficient, but rather expensive in first cost and power; several paddle wheels attached to the vertical shaft have been found to work efficiently, and are cheaper. A churn having a capacity of from 300 to 500 gal. is a convenient size. The necessary heating tanks are evidently matters of individual requirement, and require no description in detail. The process is most likely to break down through lack of efficient cooling between the churn and the storage tanks. The cooler should be of sufficient capacity to chill the material, and, of course, must be well supplied with water and have proper arrangements for heating and storage. Two men should have no difficulty in making from 8 to 10 batches per day.

Mr. Poore. H. C. Poore, Jun. Am. Soc. C. E.—The speaker's experience has been mainly with bituminous construction, but, during the past season, 300 000 sq. yd. of macadam and gravel surface have been treated under his supervision with a light refined coal-tar, for the purpose of laying the dust and incidentally preserving the road surface. A perfect dust layer should also prevent the road surface from "potting out" or ravelling.

The general method of application was as follows: After the surface had been thoroughly swept, thin refined tar (½ gal. per sq. yd.) was applied by a pressure-tank wagon, sprinkler wagon, or by some form of hand-dipper. After a few hours the surface was covered with grit. Both pea stone and fine gravel were used, and 1 cu. yd. of material usually sufficed to cover 120 sq. yd. of surface. Where the entire road or only a portion could be closed and guarded for 24 hours after spreading, the coating of grit was not always necessary. The average cost of this work in Massachusetts was from 3 to 5 cents per sq. yd., including all materials and labor.

With this treatment, a macadam or gravel road in fair condition will become hard and free from loose grit in the course of about 10 days. The surface then remains in condition for the season, and, with a re-treatment of ¼ gal. per sq. yd. in the fall or following spring, will be in good condition for a second year. The second treatment may be applied at a greatly reduced cost.

In various localities a thin asphaltic oil is laid in practically the same way on some streets, while on others, where it is desired to have a clean and hard surface which will remain in good condition through rainy and variable fall weather, refined coal-tar is preferred. Since a tarred surface is hard, and does not assimilate all foreign matter dropping on it, but easily allows the street to be cleaned by hand or by Nature, tar materials usually remain intact during the entire season, and in some cases throughout the winter.

In one large city in Maine, where refined coal-tar was used for dust-laying on a large area, but where a larger quantity per square yard was applied than is customary, a dustless surface has been in

existence for 2½ years, with no maintenance cost whatever. This is not to be considered as a heavy film treatment, as the surface was mosaic, and the tar was absorbed by the road materials. Mr. Poore.

A very efficient and economical way of applying a thin tar for dust prevention is by using 500-gal. tank wagons, fitted with auxiliary pressure tanks for blowing out the material through suitable nozzles under pressure. The use of a hose with a nozzle is a very practical method of distributing all light materials, and when they are delivered by pressure, the spreading of a ¼-gal. coat to the square yard is easily accomplished. Two applications of this quantity of material during the first year, followed by one coat during the second year, will suffice to keep the surface of a macadam road in first-class condition.

Charles W. Ross, Esq.* (by letter).—Of the palliatives used in Newton, Dustoline and various oil emulsions are of the greatest interest. Dustoline was used on about 40 000 sq. yd. of one of the main thoroughfares, and the cost for the season was less than 3 cents per sq. yd. of roadway. Mr. Ross.

The principal emulsions used on the streets included in the street-sprinkling system are "Terracolio" or "Headley's Dust Preventive No. 1," "Speare's Road Binder," and "Standard Emulsifying Road Oil." These were mixed in the proportions and rate of 500 gal. of the emulsion to 500 gal. of water, and sprinkled on the roadway from an ordinary water cart. Most of these materials last from 1 to 3 weeks, according to the weather.

SURFACE TREATMENT WITH TARS, HEAVY OILS, ETC.

Charles W. Ross, Esq.* (by letter).—The last few years have witnessed an increased use of various materials for the treatment of roads, with a view to their preservation, as well as the prevention of dust. The methods have been of two classes, namely, by improved processes of road construction, and by surface treatments. Mr. Ross.

With the advent of the automobile, new conditions confronted road engineers, and they were forced to decide at once whether they would use materials for simply suppressing dust, such as oils or emulsions which act partly as binders, or bituminous liquids which would prevent wear and disintegration by forming a water-tight wearing coat.

The continuous and fast motor traffic to which roads are subjected denudes them of the fine cementing surface material. The effect of such traffic on stone roads has been the subject of much study by highway engineers in nearly every civilized country, and many theories have been advanced to account for the injury to the roads. It has

* Street Commissioner, West Newton, Mass.

Mr. Ross. been generally agreed that the suction of the pneumatic tire on the surface is the main cause of the trouble; but in a recent address, Logan Waller Page, M. Am. Soc. C. E., advanced the theory that the great tractive force, or longitudinal shear exerted by the driving wheels of motor cars is the main cause of the injury. This is doubtless due largely to irregularities on the surface of the road. The slip, of course, increases the quantity of finely-divided surface material which is thrown into the air, and, as the amount of damage increases in proportion to the irregularities of the road and the speed of the car, the effect will be greatly reduced if the road has a smooth surface.

The prevention of dust offers two distinct problems: In the first place, the question naturally arises as to the methods to be adopted in the construction of new roads in order to bring about the desired properties of freedom from dust and low cost of maintenance. The second problem is that of the treatment of roads already in existence, to prevent wear and disintegration.

There are two general methods by which such roads are treated to remedy the dust nuisance, although it is difficult to draw a definite line between them. The first is by sprinkling the surface with any of the emulsions, and the second is by treating the road with a binding material so that only a minimum amount of dust will be formed. The first method of suppressing the dust is of necessity a temporary expedient. Water, salt solutions, light oils, emulsions, etc., come properly in this class. In the second method, it is obvious that the essential properties in a preservative are its binding power and durability. For this reason, and by way of distinction from temporary treatments, materials for this purpose can be properly called permanent binders. Such materials as tar and heavy asphaltic oils are included in this class. It must be pointed out, however, that the word "permanent" is only used in a relative sense, and that binders which on application will last at least one whole season, are included under this head.

The principal agencies which tend to cause deterioration on tarred and oiled roads are frost, heavy rains, and decaying organic matter on the surface of the road. In the writer's opinion, this last mentioned cause is more destructive than either of the others, and, for this reason, after a street is treated, it should be swept, by hand sweepers or otherwise, often enough to keep it clean. As far as can be judged from experiments, one kind of road resists the action of these agencies as well as another.

Oil, as a rule, penetrates better than tar, but its value is in the asphalt base, and, before the maximum binding power is reached, the more volatile constituents of the oil must evaporate, which is a slow process. A properly treated road should resemble sheet-asphalt pavement, although it is of a more resilient character. The surface material is all bonded together by the binding material, forming a

smooth, firm surface which can be cleaned in the same manner as a sheet-asphalt pavement. Mr Ross.

Probably as early as 1894 crude oil was used for road treatment in Santa Barbara, Cal. In 1898 six miles of road were treated in Los Angeles County, simply with the idea of laying the dust. The experiment met with astonishing success, and the practice spread rapidly. The oil not only stopped the dust nuisance, but bound together the particles of the road surface, forming a tough layer resembling asphalt pavement. Since California made these first experiments, oil has been used extensively in many parts of the country. It has been found that the quality of the oil used is of the utmost importance.

Too much stress cannot be laid on the fact that the street must be thoroughly clean before any of the material is applied. It is not necessarily advisable to have the road surface absolutely dry, practice having shown that the oil has a tendency to adhere to the dry dust on the road. This can be prevented by sprinkling the surface with water two or three hours before the work is to be done. The oil then penetrates, and prevents the pulling up of the oil and sand.

The writer has tried most of the asphalt preparations in use at the present time for preserving the road and rendering it dustless, and has classified these in three grades, as follows:

Grade A.—This is an asphaltic oil, sufficiently fluid to be applied to the roadway at atmospheric temperature, and is used on the surface of a finished road. It will effectually bind the material in the road, render it water-proof, and protect it from wear of traffic of all kinds. Automobile traffic tends to iron it out and improve it, rather than destroy it. It renders the surface resilient and elastic, and provides an agreeable tread, both for horses and vehicles.

Grade B.—This is a heavier material than Grade A. It is used for the building of new roads, or for resurfacing or reconstructing old ones, and is applied hot to the second course of stone, as set forth in the specifications under what engineers define as the "Penetration Method." It makes a solid bituminous road, extending from 2 to 3 in. below the surface, and will last and remain dustless for several years.

Grade C.—This is a semi-solid bituminous product, and is used in building or resurfacing roads by the "Mixing Process." It produces a hard, smooth, dustless roadway, absolutely water-proof, and cemented so solidly that a pickaxe cannot displace or disarrange it without great effort and labor.

At West Newton, Mass., since adopting the new methods of work, large quantities of Asphaltic Oils, Tarvia, Tarite Asphalt, and Standard Macadam Binder have been received. The materials come in tank-cars which are heated and unloaded in the following manner: The car is first connected with a small steam boiler, and heated to a tem-

Mr. Ross. perature of about 100° Fahr., thus rendering the material thin enough to flow through the hose at the bottom of the car, which rests on a trestle about 10 ft. above the surface of the ground.

When adaptable to the work, a Studebaker tank-wagon is used in applying these materials to the roadway. This wagon consists of a steel tank, holding about 550 gal., with forty perpendicular tubes, 2 in. in diameter, passing through it. Under this tank two large burners are supplied with kerosene oil forced in by pressure. The fire-box, extending the whole length of the tank, gives an even heat. The material is applied through a sprayer or spreader which is perforated so that the oil trickles down on the roadway in very fine streams, covering it thoroughly in one application. In the writer's opinion, this is a very satisfactory way of applying any heavy asphalt, as the material can be readily heated, and can be applied in large or small quantities, the flow being controlled by valves which can be regulated so that they will spread as small a quantity as $\frac{1}{8}$ gal. per sq. yd. In applying these materials it is better to close half the roadway at a time, leaving the other half open to traffic.

Asphaltoilene was used in 1907 on two roadways in Newton, a surface of about 16 822 sq. yd. being treated at the manufacturers' contract price, 6 cents per sq. yd. At present these roads are in very good condition.

Several macadam-surfaced streets, having varying grades up to a maximum of 9%, and subjected to heavy horse-drawn and automobile traffic, were selected for the liquid-asphalt treatment. The method used was as follows: A quantity of sand was heated to a temperature of about 200° Fahr., dumped in a pile, and leveled. The asphalt was poured over the hot sand in the proportion of 1 gal. to each cubic foot of sand, and then the whole mass was turned with shovels or mixed in a concrete mixer (the latter being preferable on account of cost). This work was done at the pit. The mixture was teamed to the work and spread on the roadway to a depth of less than $\frac{1}{4}$ in., being raked even with 14-tooth wooden rakes. Rolling was not considered necessary, and the street was kept open for traffic at all times. The cost of this treatment was about 3 cents per sq. yd. It has the advantage of leveling and building up the surface of the road, each new application providing a new wearing surface. This work has remained in perfect condition without further expense since the summer of 1909.

Some cities have had very good results by using the following process for patching. Gravel from an ordinary gravel pit is run through a $\frac{1}{2}$-in. screen, using a little more than $\frac{1}{2}$ gal. of the asphaltoilene, or any of the heavy oils, to a cubic foot of gravel. It is better to heat the oil or the gravel, unless the work is done in the summer, in which case there is no need of heating. A quantity of material can be prepared at the pit and kept ready for use at short notice. The

writer has found that the best time to apply this treatment is after a heavy rain, as all the depressions in the roadway show little pools of water, and if these are filled with the mixture and the surface is raked, it saves the expense of resurfacing the entire street. Mr. Ross.

The idea of applying tar to the surface of broken-stone roads probably originated with an Italian physician, Doctor Guglielminetti, the method being first applied to the streets of Monaco. The first authentic mention of a practical application of coal-tar to the surface of a completed macadam road was in 1880, when a street near Bordeaux was given a surface treatment. The results were unsuccessful, but 6 years later a similar experiment was made at Melbourne, Australia, with so much better results that the practice was continued. More care was taken in this experiment, the road surface being first cleaned, and after the tar was applied the road was given a covering of fine crushed stone. The results showed that tar had considerable value as a dust-layer by cementing the surface of the road together, thereby preventing its deterioration. The wear on the road was diminished, and consequently the cost of maintenance was very materially reduced. Since that time, other experiments have been tried, in France, England, and the United States. The use of tar on road surfaces in the United States has been confined largely to the East and North, except in a few instances where the park authorities in the larger cities have made some experiments.

One of the first experiments of any importance in the United States was made at Jackson, Tenn., in the summer of 1905. The United States Office of Public Roads, in co-operation with Mr. Samuel C. Lancaster, City Engineer of Jackson, made a series of careful experiments to determine the value of coal-tar treatments on broken-stone roads. These experiments created such widespread interest that similar ones were made in different sections of the country.

The State of Massachusetts was among the first to use Tarvia, a number of cities in the vicinity of Boston, as well as the Metropolitan Park Commission, and the State Highway Commission, treating several miles of roadway. At about this time Rhode Island and New York were using Tarvia and other tar preparations on their State roads and streets with good results. The foregoing experiments show conclusively that a specially prepared tar has given better results than any of the crude tars in use up to the present time.

In Newton the first Tarvia was used on a section of the boulevard, in September, 1906. The roadbed was first repaired and shaped up. The area covered was 3 500 sq. yd., and the cost was 14 cents per sq. yd. This section has not received any repairs up to the present time, and is apparently in first-class condition in every way.

In 1907 a section of a road, the macadam surface of which had become badly worn, was treated as follows: It was spiked up with

Mr. Ross. the roller and covered with a light grouting of 1:2 Portland cement mortar, which was spread evenly over the entire surface, and penetrated into the stone in such a way as to bind the whole material together. It was then smoothed off with large street-sweepers' brooms. The road was closed for about 4 days, giving the cement time to set. After this it was opened to traffic and used for about a year. Then the surface was swept with an ordinary horse-sweeper, and covered with a thin coating of Tarvia A, heated to a temperature of about 170° Fahr., which adhered readily to the cement, giving a smooth and satisfactory surface. This still remains in first-class condition, after a period of about 3½ years. On a small portion of the same street a second coating of cement grout was applied and covered with Tarvia A. This experiment was for the purpose of ascertaining whether the materials would adhere and form a satisfactory surface. It has proved to be all that was expected.

In recommending the use of the different tar treatments, it is well to bear in mind that they are better adapted to macadam than to gravel surfaces, because the surface is likely to become broken unless the foundation is solid enough to carry the load, and, once broken, deterioration is very rapid. The asphalt oils, however, can be applied to either. They stand the winter weather without breaking, and in the spring are as smooth as when built.

Within the past two or three years, or since the automobile came into general use, many engineers have almost come to the point of deciding against the construction of macadam roads. While this may be true as relates to water-bound macadam, yet, in view of the small cost of building and maintenance, as compared with the more expensive forms of laid pavements, it is questionable if anything better than macadam has yet been discovered for modern traffic in towns and small cities, provided it is bound or surfaced with bituminous preparations. Experience has shown that if a macadam road is properly treated when first built or extensively repaired with any of the foregoing bituminous binders, the results will be satisfactory; and, considering the difference in cost between the old form of water-bound macadam and the new method of using bitumen, the amount is very slight compared with the advantages of a roadway which is water-proof, dustless, easily cleaned and repaired, and will endure both horse-drawn and mechanically propelled vehicles. The automobile is not so much to be dreaded as was at first supposed. A road properly treated will stand the traffic of automobiles and horse-drawn vehicles and wear for as many years as it would without mechanical traction.

The progress which has been made in the past 10 years is very gratifying. The time has now arrived when engineers must require as effective construction as possible, which practice is always the cheapest in the long run.

W. W. Crosby, M. Am. Soc. C. E.—During 1910, under the direction of the speaker, about 50 miles of road in Maryland were treated with various tars and heavy oils, most of this work being on newly finished water-bound macadam. For the sake of conciseness, the information concerning this construction is submitted in Table 1. Mr. Crosby.

For the road between Baltimore and Washington, portions of which had been improved with water-bound macadam in previous years, the information is given concisely in Table 2.

The traffic over the roads mentioned in Table 1 has been counted and is given in Table 3.

The traffic over the sections of the Baltimore-Washington Road has been counted, and is given in Table 4.

In September, 1908, a 3 year-old stretch of water-bound, traprock macadam, in fair condition, 12 ft. wide and 538 ft. long, was swept thoroughly, and coated with refined water-gas tar to the extent of 1.08 gal. per sq. yd. A coating of stone chips was then applied in sufficient quantity to take up any excess of tar which did not penetrate the road itself in about 5 hours. The chips were then well rolled. The cost was as follows:

Pitch (at 6 cents per gal. delivered)	6.6	cents per sq. yd.
Stone chips (at $1.75 per ton delivered)	2.3	" " " "
Labor and incidental expenses	3.8	" " " "
Total	12.6	cents per sq. yd.

This section has been, and is now, in an entirely satisfactory condition; it has not needed any attention or expense since construction. The traffic on this road, as counted recently, was:

Single vehicles	66	Average traffic per day of 12 hours.
Double vehicles	9	
3- (or more) horse vehicles	8	
Motor cycles	3	
Runabouts	3	
Touring cars	41	

It is impossible to give the ultimate cost of the above work now, as the data when re-treatment will be necessary are unknown and at present not even in sight.

In addition to the foregoing, a material, which might perhaps be included as a "Heavy Oil," for the purpose of reporting on it under this topic, has been tried.

In November, 1909, a section of Park Heights Avenue, about 6 miles from the City Limits, was treated by simply spraying "Glutrin" from an ordinary sprinkling cart on the surface. The quantity used was 1 600 gal. on 2 300 sq. yd. of water-bound limestone macadam which

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TABLE 1.

Road, reference No., and when done.	Length, in miles.	Width, in feet.	Kind of material in macadam.	Material used for surface treatment, and quantity per square yard.	Covering used and quantity per square yard.	Total cost of treatment, per square yard, or contract price.
Forestville-Marlboro, Contract No. 0130, November, 1910	3.19	14	Limestone.	U. G. I. dust-layer 0.527 gal.	Sand. Pea gravel. 10.83 lb.	$0.049
Greensboro-Denton, Contract No. 050, November, 1910	2.59	14	Trap rock. Limestone screenings.	Asphaltoilene. 0.518 gal.	Sand. Pea gravel. 18.11 lb.	0.056
Denton-Federalsburg, Contract No. 051, November, 1910	1.15	14	Trap rock. Limestone screenings.	Asphaltoilene. 0.597 gal.	Sand, Gravel. 18.64 lb.	0.066
Hickory, Contract No. 0175, October, 1910	2.38	14	Trap rock.	Texaco special. 0.6874 gal.	Roland Park Gravel. 9.40 lb.	0.086
Forest Hill, Contract No. 0171, October, 1910	1.93	14	Trap rock.	Texaco special. 0.509 gal.	Roland Park Gravel. 9.49 lb.	0.066
Easton-Wye Mills, Contract No. 0110, August, 1910	3.03	14	Limestone.	Asphaltoilene. 0.492 gal.	No covering.	0.05*
Federalsburg-Hurlock, Contract No. 052, September, 1910	1.12	14	Limestone.	Standard Oil No. 5. 0.519 gal.	Sand. 7.84 lb.	0.03
Federalsburg-Hurlock, Contract No. 070, September, 1910	5.36	14	Limestone.	Standard Oil No. 5. 0.504 gal.	Sand. 11.36 lb.	0.028

* Contract price.

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TABLE 2.

Section, and when macadamized.	Length, in miles.	Width, in feet.	Material of macadam.	Cost of repairs to macadam, per square yard.	Kind of material used for surface treatment, and quantity per square yard.	Covering used, and quantity per square yard.	Cost of surface treatment, including covering, per square yard.	Total cost of repairs and treatment, per square yard.
Section No. 9....... 1909	2.76	14	Limestone.	$0.0176	Standard No. 5. 0.497 gal.	Pea gravel. 18.49 lb.	$0.029	$0.047
Section No. 7....... 1909	1.80	14	Limestone. Trap rock.	0.0526	Standard No. 5. 0.442 gal.	Pea gravel. 18.31 lb.	0.036	0.088
Section No. 5....... 1908	2.30	14	Trap rock. Gravel.	0.0496	Standard No. 5. Fairfield Anti-Dust. 0.54 gal.	Pea gravel. 13.26 lb.	0.033	0.083
Section No. 1....... 1906	0.53	12	Trap rock.	0.0165	Standard No. 5. 0.503 gal.	Sand. 12.05 lb.	0.025	0.19
Section No. 2A..... 1909	0.19	16	Trap rock.	0.0145	Standard No. 5. 0.6725 gal.	Sand. 12.30 lb.	0.034	0.048
Section No. 4A..... 1910	0.78	14	Trap rock.	0.0245	Standard No. 5. 0.547 gal.	Sand. 13.33 lb.	0.031	0.056
Section No. 8A..... 1907	1.63	14	Limestone.	0.034	Asphaltoilene. 0.564 gal.	....	0.05	0.084*
Section No. 8B..... 1909	1.57	14	Limestone.	0.0704	Asphaltoilene. 0.564 gal.	....	0.05	0.057*
Section No. 12B.... 1909	1.59	14	Limestone.	0.0225	Asphaltoilene. 0.564 gal.	....	0.05	0.072*
Section No. 12A.... 1908	0.97	14 16 18	Limestone.	0.0321	Asphaltoilene. 0.564 gal.	....	0.05	0.082*
Section No. 14A.... 1909	0.44	18	Limestone.	0.0156	Asphaltoilene. 0.564 gal.	....	0.05	0.065*
Section No. 3....... 1908	2.61	14	Trap rock.	0.104	Standard No. 5. 0.457 gal.	Sand and gravel. 26.046 lb.	0.028	0.132

* Pitching done by contract.

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was built in 1908, and in November, 1909, was still in very good condition. The limestone was of the white variety, rather soft, though highly cemetitious. The "Glutrin" was spread about 1 ft. beyond each edge of the 12-ft. wide macadam, as the shoulder material was ordinarily of poor quality. The traffic over this section of road had been fairly light, and, until the fall of 1910, the "Glutrin" treatment was visible and seemed to have had some good effect. The road surface of the treated section was less friable and less dusty than the untreated macadam of the same road immediately beyond. The cost of this "Glutrin" treatment in 1909 was about 10 cents per sq. yd. During 1910 a branch road was opened into Park Heights Avenue at about the middle of the "Glutrin"-treated section, and hence the traffic over a part of it greatly increased.

TABLE 3.—AVERAGE TRAFFIC PER DAY OF 11 HOURS.

Reference number of road.	One-horse.	Two-horse.	Three-horse.	Four-horse.	More than four-horse.	Motor cycles.	Runabouts.	4- or 5-seat cars.	6- or 7-seat cars.	Motor drays.
Forestville-Marlboro. Contract No. 0130	27	19	0	0	0	0	0	0	0	0
Greensboro-Denton. Contract No. 050	132	32	0	2	0	1	3	4	0	0
Denton-Federalsburg. Contract No. 051	213	36	0	2	0	2	7	10	0	0
Hickory. Contract No. 0175	43	9	0	3	1	1	0	0	0	0
Easton-Wye Mills. Contract No. 0110	197	70	0	0	0	1	4	2	0	0
Federalsburg-Hurlock. Contract No. 052	54	10	0	0	0	0	4	4	1	0
Federalsburg-Hurlock. Contract No. 070	54	10	0	0	0	0	4	4	1	0

TABLE 4.—AVERAGE TRAFFIC PER DAY OF 10.5 HOURS.

Section number.	One-horse.	Two-horse.	Three-horse.	Four-horse.	Six-horse.	Motor cycles.	Run-abouts.	4- or 6-seat cars.	7-seat cars.	Wagons.
9	115	25	1	0	0	20	3	29	11	1
2-A	151	26	0	1	1	1	1	7	3	1
4-A	71	12	0	0	0	0	2	6	0	0
12-B	88	33	0	2	0	4	3	19	4	0
12-A	233	98	2	2	0	3	6	20	10	0
14-A	267	142	3	4	0	7	4	33	5	0

The following is a recent count below the branch road referred to:

One-horse	12	Average traffic per day of 12 hours.
Two-horse	1	
Four-horse	5	
Six-horse	13	
6- or 7-seat cars	1	
Omnibuses or drays	1	

It was decided to treat the main road below the branch road with bituminous material, which was done. The remainder of this road, to

its end (about 10 000 ft.), needed some repairs to the macadam. These were made by water-binding methods, at an average cost of $0.276 per sq. yd., calculated on the basis of the total area of the macadam in the road, as the road was built in 1908 and had received practically no maintenance. Then "Glutrin" was applied to the macadam through an ordinary sprinkling wagon. Mr. Crosby.

The proportions used were 5 gal. of "Glutrin" to 2 gal. of water, or 0.426 gal. of "Glutrin" per sq. yd. of macadam. It was sprinkled about 2 ft. outside the edges of the macadam. The work was completed about December 1st, 1910. The cost was as follows:

Repairing macadam	$0.276 per sq. yd.
Treating with "Glutrin"	0.07 " " "
Total cost	$0.346 per sq. yd.

The traffic on this section, as counted recently, was:

One-horse	15	Average traffic per day of 12 hours.
Two-horse	4	
Four-horse	2	
Six-horse	5	
Runabouts	2	
4- or 5-seat cars	10	
7-seat cars	2	

Mr. Hubbard.

Prevost Hubbard, Esq.*—Without doubt, the most vital question to be considered in connection with the surface treatment of macadam roads with heavy oils is: Under what conditions will such treatment prove satisfactory? Apart from the condition of the road surface before treatment, the character of the oil, and the method of application, a most important factor is the traffic to which the road is, or will be, subjected. Considerable attention has recently been paid to the first three, but the last seems to have been overlooked to a great extent, at least, in so far as available data are concerned. It is to this factor—the effect of traffic on macadam roads surfaced with heavy oils—that the speaker wishes to call particular attention.

Granting that a given road in good condition has been treated properly with a good grade of oil, experience indicates that horse-drawn traffic will damage the surface mat of oil and stone screenings to a greater extent than automobile traffic. In fact, the latter, if not excessive, seems in certain ways to be beneficial to such a surface, by keeping it smooth and well compacted. Horse-drawn traffic, on the other hand, tends to cut and roughen the oil mat, and if the ruts and caulk marks are not ironed out promptly by the passage of motor vehicles, the mat is apt to disintegrate and produce an oily, disagreeable

* Chemist, Office of Public Roads, Washington, D. C.

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mud in wet weather. In general, it may be said that such damage is most pronounced during cold, rainy weather, and that an oil-surfaced road will prove most unsatisfactory at such times. This being true, particular attention should be paid to traffic conditions during bad weather.

In ascertaining from previous experiments under what conditions surface treatment will prove satisfactory, the following facts are most necessary:

(1) The volume of traffic in bad weather;
(2) The proportion of horse-drawn vehicles to motor vehicles during bad weather;
(3) The character of both classes of traffic during bad weather.

A certain instance has come under the speaker's observation: Two macadam roads, in good condition, were treated in the same manner, and with the same oil, as shown by analysis. One of these roads proved to be quite good throughout the year, while the other was far from satisfactory during bad winter weather, at which time an oily mud was formed over the surface to a depth of about 2 in. Eventually, this mud had to be removed. The oil was a heavy semi-asphaltic residuum, barely fluid enough to apply cold in a satisfactory manner. The following are the specifications for this oil:

SPECIFICATIONS FOR ROAD OIL.

This oil is to be used cold in the surface treatment of clean, well-consolidated macadam roads.

(1) The oil shall be a viscous fluid product, free from water, and showing some degree of adhesiveness when rubbed between the fingers.

(2) It shall have a specific gravity of not less than 0.940 at 25° cent.

(3) It shall be soluble in carbon bisulphide, at air temperature, to at least 99.0%, and shall show not more than 0.2% of inorganic matter insoluble.

(4) It shall contain not less than 3.0%, nor more than 10.0%, of bitumen insoluble in 86° Baumé paraffin naphtha at air temperature.

(5) When 240 cu. cm. of the oil are heated in an Engler viscosimeter to 50° cent. and maintained at this temperature for at least 3 min., the first 50 cu. cm. shall flow through the aperture in not less than 10 min., nor more than 20 min.

(6) When 20 grammes of the material are heated for 5 hours in a cylindrical tin dish, approximately 2½ in. in diameter and 1 in. high, at a constant temperature of 163° cent., the loss in weight by volatilization shall not exceed 20 per cent. The residue should be decidedly sticky.

(7) Its fixed carbon shall be not less than 3.5 per cent.

A traffic census of the two roads, taken on various days during bad weather, showed that it was quite uniform at such times. For both, the traffic consisted mainly of automobiles, light carriages, and

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delivery wagons, no heavy trucking being allowed. The maximum traffic on the road in good condition amounted to 41 vehicles per hour, while on the road in bad condition there were 127 vehicles per hour, giving a proportion of about 1 to 3. This proportion was practically the same for each hour taken. On the road in good condition, 5 automobiles were counted for every 4 horse-drawn vehicles, while on the bad road the proportion of automobiles to horse-drawn vehicles was approximately 1 to 2. In view of the fact that on neither of these roads can the volume of traffic be considered as excessive, it seems probable that the difference in results can be attributed directly to the greater proportion of horse-drawn vehicles to motor vehicles.

Of course, too great reliance should not be placed on a single set of observations, but the following facts would seem to be indicated from these experiments:

(1) With the type of oil used, satisfactory results may be expected when the automobile traffic is in excess of the horse-drawn traffic during bad weather;

(2) With this same oil, unsatisfactory results may be expected when the proportion of horse-drawn vehicles is double that of motor vehicles during bad weather.

In the speaker's opinion, the danger line lies much closer to the point where the proportion of horse-drawn vehicles just exceeds that of motor vehicles during bad weather, but this remains to be proved. It would seem reasonable, however, to expect that the number of automobiles must be equal to the number of horse-drawn vehicles in order that the marks on the oil mat made by the latter may be ironed out as fast as formed.

The limited time necessarily allowed for these discussions makes it impossible to enter into as much detail as might be desired, and therefore the speaker will conclude with the suggestion that in future surface treatments, careful records of traffic during bad weather be kept, in order to determine when such treatment can be expected to prove satisfactory. This information should be particularly valuable to those who have occasion to treat park and pleasure drives, as surface treatment would seem to be of the greatest value on roads of that type.

Mr. Blanchard.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The extent and the success of the future use of surface treatments in the United States depends on the development of economical and efficient methods for the distribution of bituminous materials, the use of methods and materials adaptable to local conditions, the construction of macadam and gravel surfaces in such a manner as to be suitable for the effective superficial application of bituminous materials, and the adoption of an efficacious system of continuous maintenance, including the installation of proper plant equipment. That superficial treatments are entirely satisfactory

Mr. Blanchard. from the standpoints of economics and efficiency when the above conditions are fulfilled, the practice of English and French engineers during the past five years clearly demonstrates.

The present seems to be an opportune time to discuss some of the disputed points relative to methods and materials used in connection with surface treatments. Not only is there a decided difference of opinion among American engineers as to the value of various methods and materials, but foreign engineers—especially the English and French—also hold widely divergent opinions with reference to certain details. Among the points which should engage the attention of engineers are the following: The relative advantages of pressure and gravity-flow distributors; the amount of penetration of an ordinary macadam road obtained by the use of a pressure spraying machine; the relative advantages of sand, shingle, gravel, and stone chips as a covering; the feasibility of the omission of the covering; the practicability of drying a damp macadam surface artificially in order to render possible the application of such a bituminous material as tar, thus obviating the unnecessary expense due to waiting for the road to dry out; the relation of the size of stone composing the surface of a macadam road to the economics and efficiency of surface treatments; the slipperiness of surfaces treated superficially with various bituminous materials under varying climatic conditions; the effect of non-skidding devices, particularly chains, steel studs, and rubber blocks, on various surfaces; and cost data covering a series of annual applications of a given kind of bituminous material on the same road.

Mr. Eppele. FRANK J. EPPELE, ESQ.* (by letter).—It is not intended to enter into a technical discussion of the relative values of the various methods of surface treatment or even to consider the comparative merits of the numerous compounds used, but to confine the discussion to a review of the work done by Mercer County, New Jersey, in maintaining its system of improved highways.

That work has been surface treatment, and has been confined almost entirely to the application of a bituminous dust layer known as "No. 4 Road Oil," guaranteed to contain from 40 to 45% of petroleum asphalt, and "No. 5 Road Oil," having an asphaltic content of from 50 to 55 per cent. The primary object in using this oil compound was to allay dust, but it was also found that its application added materially to the condition of the surface of the road, and, for the time being, obviated the necessity of making more costly repairs. It was demonstrated that on macadam roads, where a slight disintegration of the surface had begun, further destruction could be retarded and the road maintained through another season in good condition by a single application of road oil with the addition of a thin coat of stone screenings. Relatively better results were secured when two applications were made.

* County Engineer of Mercer County, N. J.

Mr. Eppele.

The oil was purchased by the car load. Without heating, it was distributed on the road with an ordinary sprinkler having an oil-distributing attachment. The cost, including the screenings, was about 8 cents per sq. yd. This work, while giving good results, was not entirely satisfactory on account of the uneven distribution of the oil caused by variations in the speed of the horse-drawn sprinkler. These speed variations were due to change of grades and the passing from a maximum to a minimum load to be hauled, and also to the uneven oil pressure maintained at the distributing point on account of the constantly changing head. This objectionable feature was entirely eliminated by the use of an auto-sprinkler owned by the oil company. With this sprinkler perfectly satisfactory results were obtained, the machine being capable of traveling at uniform speed, regardless of changes in grade or load. The varying pressure due to head of oil was overcome by maintaining uniform air pressure in the tank. This pressure added materially to the final result by forcing a portion of the oil into the body of the road. Almost double the quantity of oil can be used without causing small rivulets to flow to the sides of the road when the distribution is accomplished in this way.

After this first trial, all surface oiling was done with auto sprinklers at a cost of 1.2 cents per sq. yd. of surface treated; but this did not include the cost of the coat of screenings. The cost of furnishing and spreading the necessary quantity of screenings was 3 cents per sq. yd., thus giving a total cost of 4.2 cents as compared with 8 cents per sq. yd. when the entire work was done by the County.

This method of surface treatment, although of a temporary nature only, gave very good results, and, when considered in connection with its cost, deserves to rank among the foremost methods of maintaining an already constructed water-bonded macadam road. Great care should be taken, however, in selecting an oil for surface application. If the oil is too thick, it will remain on the surface and will not penetrate the body of the road. When of this consistency, it will adhere to the wheels of passing vehicles, causing more or less discomfort to the traveling public as well as injury to the road surface. On the other hand, if the oil is too thin or too light, it has too great a spreading and penetrating power for the base it contains. The result secured with such oils is a stained surface more or less devoid of any cementing bitumen, making frequent applications necessary. An ideal oil is one which will penetrate and yet have sufficient base to assist in holding together the mineral aggregate. In the writer's judgment, a product containing from 40 to 45% of petroleum asphalt will generally give the best results.

Of course, it is presumed that the surface of the road to which the oil is to be applied is in fair condition, being free from ruts or other pronounced inequalities. If it is not in fair condition, the surface should be renewed before the application is made. In fact, no

Mr. Eppele. method of surface renewal should be used for roads in poor condition unless provision is made for spreading a mineral aggregate to a depth of 3 in. or more.

Another method of surface treatment, which gives every indication of producing good results, is the application of a heavy residuum oil, having a specific gravity of about 12° Baumé and a residue of about 90% when reduced to 100 penetration at 77° Fahr., which, when absorbed by the sand coating spread over the oiled surface to a depth of 1 in. or more, depending on the quantity of oil to be taken up, forms a sand asphalt coating which is practically water-proof, dustless, and noiseless, and might be termed an artificial rock asphalt. Unlike the oil product previously referred to, this asphaltic compound, while having no penetrative qualities whatever, possesses distinct binding qualities. It is obtained by the distillation of an asphalt-base petroleum oil.

The method of applying this oil was as follows: The surface of the road was first carefully swept to remove all loose matter, after which water was applied with a sprinkler in sufficient quantity to saturate the road thoroughly, the object being to remove any loose particles remaining on the surface and to deposit them in the existing crevices. Uniform distribution of the oil cannot be accomplished if there is much loose material on the surface. The oil was received in tank cars, and, after heating to about 150° Fahr., was pumped into a road sprinkler and applied to the washed surface with a special spreading device, using ½ gal. per sq. yd. of surface. When the asphaltic product came in contact with the wetted surface of the road, it was chilled, thus preventing it from running off at the sides. While in this chilled state, it was absorbed by the sand coating which had been immediately spread. The sand used was of a sharp, gritty nature, entirely free from loam and moisture. The road was opened to traffic after a reasonable amount of rolling, and the spreading of additional sand at points showing a surplus of oil. The condition of the surface then improved daily. The value of this method of repairing roads depends on the fact that an oil of this consistency, when covered with a layer of sand, will be slowly drawn to the surface by solar heat; consequently, work of this class should be done only on clear, warm days.

The experimental work done by this method, was necessarily crude, owing to improper machinery, etc., but it is understood that heater tank wagons are now being used which apply the oil under pressure, and thus give more uniform results by eliminating the probability of having a varying thickness of resurfacing material.

It is essential to coat every part of the road with a layer of oil of uniform thickness, as points not coated will have no covering, and those receiving an excess of compound will become wavy by the rolling of the surface under extreme loads. It is also apparent that the

surface of a road to be treated by this method should be free from inequalities, thereby removing all possibility of water collecting in pools on the surface. Although no experiments have been made along this line, it is believed that if a light coating of pea-size broken stone is applied first, and then covered with sand, the result will be a great improvement, giving a better and more lasting covering, which will reduce to a minimum the tendency to creep under heavy loads. The asphaltic oil-sand covering which has been in use for one year indicates that fairly good results will be obtained for another season. It is proper to assume, then, that, under normal conditions, the life of roads resurfaced by this particular method is 2 years, after which, with the addition of a lighter coating, it is claimed that the life can be extended to 4 years. The cost of the compound was 5 cents per gal., delivered, and the total cost for work of this class was about 9 cents per sq. yd., or an average of 4½ cents per sq. yd. per year. Mr. Eppele.

By far the greater part of all resurfacing work on roads in New Jersey during 1910 was done, or at least started, under a State specification, which called for the spreading of 4 in. (loose measurement) of 1½-in. broken stone; the coating of the stone by the application of heated asphaltic binder to the surface; thorough rolling; the addition of ¾-in. stone, or screenings, and sometimes sand, and continued rolling until the road was finished. Although it was thought that excellent results would be produced by this method, it failed to come up to expectation.

A large number of contracts for new construction, as well as reconstruction work, had been entered into throughout the State, when, in the middle of the season, it was decided, with the consent of all interested parties, to change the contract by eliminating the use of the asphaltic binder specified and by providing for the construction of a water-bonded macadam, and the application, when completed, of "No. 4 Road Oil" to the surface. In every case this change carried with it an allowance or rebate, on the part of the contractor, to the State and County, and, at the same time, gave greatly improved results, when compared with those previously obtained.

That part of the original specification relating to the qualities of the asphaltic binder to be used was prepared by an expert chemist, and called for heating the compound, which was shipped in tank cars, to 200° Fahr., after which it was to be applied to the road by a sprinkler.

It is claimed, by the parties who furnished the asphaltic binder, that the cause of failure was due, not to the chemical proportions, but to the physical properties of the compound, and that if a hard binder, without being "cut back" or fluxed with a distillate, had been specified to be applied by hand after heating to 350° Fahr., the final results would have been entirely satisfactory. This claim by the oil company, however, is not in accordance with the results obtained in New Jersey

Mr. Eppele. during the past 3 years. The binder used during 1909 was not a "cut back" product, yet the results were worse than those secured by the "cut back" compound specified in 1910. Both products were of about the same consistency. According to the opinion of the chemist referred to above, one of the causes of failure was the use of an oil having a paraffine base, oils of this type appearing to lose their binding properties much more quickly than those having an asphaltic base.

It is possible, however, with a proper asphaltic binder, to make a satisfactory bituminous macadam road by the foregoing methods, as shown by some of the roads constructed; but, to secure these results, it is essential to have experienced men, a satisfactory distributor, and a reliable contractor. A hard binder, heated to 350° Fahr., and applied by hand, should not fail to give good results.

Although the results obtained after changing the method of construction, as related above, were pleasing, it should not be understood that, in the opinion of the State Road Department of New Jersey, or the various county engineers, this is the most desirable method for future construction work. It is the writer's opinion that a bituminous binder of some kind should be used, and that a change to the old water-bonded macadam, even if it provided for the application of a light dust-laying oil to the surface, would be a step backward.

The State Road Department, however, was in a peculiar position, which, in justice to it, should be mentioned. It is understood that, in an opinion given by the Attorney-General of the State to the Road Department at the time a change of method of construction was being considered, it was held that the Department would be justified in modifying the specifications where the sole object was to improve the results after construction, provided such change did not involve an addition to the original cost; but that a change at an extra cost would not be legally justifiable without canceling existing contracts and re-advertising for bids. Under these circumstances, there was no other course to follow than the one adopted.

The average cost of resurfacing work under the original specifications was 58 cents per sq. yd., which cost included the necessary work to be done on shoulders. Deducting the average rebate of 6 cents per sq. yd. leaves a net cost of 52 cents per sq. yd. for work of this class.

Mr. Rablin. JOHN R. RABLIN, M. AM. SOC. C. E. (by letter).—In the consideration of the subject of macadam and gravel road construction and maintenance, greater importance is generally given to the problems of construction than to those of maintenance. Although the methods used and the results obtained in the construction of the road surfaces have, without doubt, great bearing on their future maintenance, nevertheless, the efforts and expense which are put into the proper construction are of little value from the standpoint of the durability of the surface, if the maintenance work is neglected.

The problems of maintenance have become very much more difficult during the past five years, and especially that of the preservation of the road surfaces constructed without bituminous binders to serve the conditions of traffic of eight or ten years ago. It has been demonstrated, however, that it is not absolutely necessary that the surfacing shall be constructed with bituminous binders in order to withstand the traffic conditions of to-day, as many miles of roadway surfaced with ordinary macadam and gravel, by early and constant attention to maintenance, have been kept in good condition for the last five or six years by using surface treatments with bituminous materials, and without any extensive work of resurfacing. Mr. Rablin.

In surface treatment there is no doubt that some advantage and benefit is obtained if the roadway surface is constructed with a bituminous binder, and such a surface can probably be maintained more economically, as more wear can be allowed without danger of disintegration, making the re-treatments less frequent and requiring less material. The results obtained on work of surface treatment with tars and oils which has been under the direction of the writer, have been generally very satisfactory, on ordinary macadam and gravel surfaces, and on those constructed with bituminous binders.

The work referred to was begun in August, 1906, when about 67 500 sq. yd. of macadam roadway surface were covered with Tarvia. Previous to this work, very little expense had been necessary on these roads, except the cost of watering, as they had been built only from four to five years. The cost of watering alone averaged about 3½ cents per sq. yd. of surface per year. The cost of this first work of surface treatment with Tarvia averaged 6⅔ cents per sq. yd., including labor and materials. The details of this cost are as follows:

Tarvia	$0.0262	per sq. yd.
Stone screenings	0.0184	" " "
Labor	0.0220	" " "

The labor item includes preparing the roadway surface and applying the tar and screenings.

When it is taken into consideration that this work acted as a substitute for re-surfacing, which would otherwise have been necessary, and also eliminated the sprinkling with water, it is evident that it was an economical method of road preservation, provided it would be effective for at least one year. That this has proven to be a fact will be shown by the costs of maintenance of this and other sections from the time of the first treatment to the present. For the following year, 1907, the total cost of repairs and re-treatments of the Tarvia section averaged 3½ cents per sq. yd. for the 67 500 sq. yd. During this year about 150 000 sq. yd. additional macadam road surface were

Mr. Rablin. treated in the same manner at a cost of from 5¾ to 9 cents per sq. yd., or at an average cost of 6½ cents, including labor and materials.

Besides this additional Tarvia work, experiments were made with other materials during 1907. About 225 000 sq. yd. of roadway—one-quarter being gravel-surfaced and three-quarters macadam-surfaced—were covered with a mixture of crude water-gas tar and asphaltic oil. These materials were mixed in different proportions of 2, 3, and 4 bbl. of oil to 6 bbl. of tar. The results obtained with this material were more in the nature of dust suppression than surface preservation, as it contained but little actual binding properties. Better results were obtained with it on the gravel surfaces than on the macadam. The average cost was 4 cents per sq. yd., including materials and labor for one application.

Asphaltoilene, an asphaltic oil, was used on about 35 000 sq. yd. of roadway, one-half macadam- and one-half gravel-surfaced. This material was furnished and applied for 6 cents per sq. yd., the only other expense being the sand covering, which was very small. The results obtained by the use of this material were very satisfactory and economical, as in the case of the gravel-surfaced road it was effective for two years, with practically no expense for repairs during that period, and the macadam roadway was maintained during the second year by patching and some slight re-treatments, at an average cost of 3 cents per sq. yd.

During the next year, 1908, about one-half of the total area previously surfaced with refined tar, or 120 000 sq. yd., was retreated with the same material, at an average cost of 4¾ cents per sq. yd., and the remainder kept in good condition without repairs. About 60 000 sq. yd. additional, of this same class of work, were done, except that a different brand of refined tar was used. The average cost of this additional work was 5 cents per sq. yd. About one-quarter of the area surface-treated during the previous year with asphaltic oil was given a light re-treatment at a cost of about 3 cents per sq. yd., and 75 000 sq. yd. additional were laid, at a cost of 7 cents per sq. yd. Of the area previously treated with water-gas tar and asphaltic oil, 53 000 sq. yd. were re-treated with the same material, and 46 000 sq. yd. additional were laid. The cost of both averaged 3½ cents per sq. yd.

A new combination of materials, which has been used to a considerable extent since, was tried in this year on about 30 000 sq. yd. of macadam surface. It consisted of 90% refined tar and 10% residual asphalt. The cost of this work averaged 5½ cents per sq. yd. The results obtained with this material appeared to be superior in some respects to those with the tar alone.

During 1909 and 1910, the tar surfaces have been maintained in good condition generally by re-treatments with the tar and asphalt mixture, at the rate of about once in two years, at an average cost

of 6 cents per sq. yd. for each re-treatment, and the asphaltic oil surfaces have required about the same attention and at the same average cost. Where traffic conditions are extremely severe, more frequent repairs are required, and the average cost will vary up to 6 cents per sq. yd. per year. Where the macadam surfacing is constructed with a bituminous binder, the surface, or painting coat, is also required from time to time, but, as stated previously, not as frequently. Mr. Rablin.

It appears from these tests of the past five years that surface treatment of macadam and gravel road surfaces with refined tars and asphaltic oils has been, up to the present time, an effective and economical method for the preservation of road surfaces subjected principally to an automobile traffic, which was the class of traffic under which these tests have been made.

R. K. COMPTON, ESQ.*—The following is a description of the method used in Baltimore of applying a surface treatment of oil, and its cost. Mr. Compton.

Where the original road-bed was in good condition for any material distance, and required no new stone for resurfacing, the surface treatment was tried. The old road-bed was thoroughly cleaned with wire brooms, and all dust, dirt, and foreign material were removed. A coating of Fairfield Anti-Dust, a local material, heated to a temperature of 190° Fahr., was then applied, to the extent of ½ gal. per sq. yd. of surface, and was thoroughly broomed in. About two hours was allowed for the bitumen to set and penetrate, after which the surface was covered with a layer of granolithic from ⅝ to ¼ in. in size, and then thoroughly rolled with a 12-ton roller.

The cost of flushing 3 024 sq. yd. of surface was 13.29 cents per sq. yd., itemized as follows:

Labor	5.00 cents.
Stone	3.00 "
Rolling	1.50 "
Binder	3.15 "
Coal	0.35 "
Hauling binder	0.15 "
Incidentals, including supplies and repairs to plant	0.14 "
Total	13.29 cents.

The surface treatment was only used where the original section was in fairly good condition, in order to prevent raveling, and at the same time to allay the dust nuisance. A recent inspection of the road, after it had been subjected to a heavy fall of snow, a sudden thaw, and freeze, in succession, showed that the surface treatment has

*Assistant City Engineer, Baltimore, Md.

Mr. Compton. served the purposes for which it was intended. Where the machines had worn a rut through the snow and ice to the surface, the road showed some signs of raveling, but this defect can be corrected at small expense in the spring. Every machine and vehicle followed this same rut for weeks, so that there must have been an enormous wear on the surface for a width of only about 6 in.

Mr. Connell. WILLIAM H. CONNELL, ESQ.*—A surface treatment of oil and sand was used in connection with experiments on White Plains Avenue, Borough of the Bronx. The description and detailed cost follow:

WHITE PLAINS AVENUE EXPERIMENTAL PAVEMENTS.

SECTION EIGHTEEN: 39 + 22 to 41 + 20.

Standard Oil Company's Sand-Surface Method.

Application of Oil and Sand.—This part of the road was an old strip of water-bound macadam. It was swept, and then Standard Oil Company's Binder A was applied, the quantity being ¾ gal. per sq. yd. Over this was spread a thickness of about 1 in. of sand, which was lightly rolled.

COST PER SQUARE YARD.

Sand course	$0.178
Hauling binder	0.002
Tar heater	0.001
Fuel	0.021
Total	$0.202

SAND COURSE.

		Per square yard.
Labor	$40.45	$0.086
Sand, 10 cu. yd., at $1.21	12.10	0.026
Binder, 384 gal., at $0.08	30.72	0.066 = 0.82 gal. per sq. yd.
467 sq. yd., at $0.178	$83.27	$0.178

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	14	$4.00	$7.00
Cleaning macadam	3	24	2.25	6.75
Pouring and carrying binder	4	40	2.25	11.25
Heating binder	2	20	2.25	5.62
Spreading sand	3	21	2.25	5.90
Tamping	1	2	2.25	0.56
Rollerman	1	6	4.50	3.37
467 sq. yd., at $0.086				$40.45

* Assistant Commissioner of Public Works, Borough of the Bronx, New York City.

The actual time of laying the sand course was $2\frac{1}{2}$ days (gang time), or an average of 187 sq. yd. per day, including cleaning and brooming old macadam. Mr. Connell.

TEMPERATURE, WEATHER, ETC.

Pavement.	Date, 1910.	Temperature.			Weather.
		9 A. M.	12 NOON.	3 P. M.	
Standard Oil-Sand...	October 18th. Begun.	59°	65°	66°	Clear.
	October 19th	59°	65°	68°	Clear.
	October 21st. Finished	55°	57°	59°	Cloudy.

THE USE OF BITUMINOUS MATERIALS BY PENETRATION METHODS.

W. W. CROSBY, M. AM. SOC. C. E.—Up to the present time, local conditions have seemed to require the use of the penetration method in the treatment of the State roads of Maryland with bituminous materials. The speaker will report only on the work directly under the supervision of the State authorities. Mr. Crosby.

In 1909 Park Heights Avenue was treated with 15 different bituminous materials in 29 sections. The details of the treatments were reported to the Second International Road Congress, and this report has been reprinted in some American publications, so that repetition will be avoided here. Heretofore, circumstances have prevented giving the costs of the 1909 work, therefore tables of cost data are submitted herewith.

The method generally used was to put down the macadam in the ordinary way up to the point of beginning the binding, as usually practiced, with water and stone chips or screenings. Slight variations from this method have been made. For example: In a few instances the courses of the macadam have been reversed and the "Number ones" placed on top, and the "Number twos," as ordinarily called, placed in the bottom. In some cases, before the pitch was applied to the stone (which had been thoroughly rolled), a layer of sand, to fill partly the voids between the stones, was put on and rolled in. In other cases the pitch was applied to the clean rolled second course.

The quantity of pitch used has varied from $\frac{3}{4}$ gal. to as much (in one particular instance) as 5 gal. per sq. yd. The use of 5 gal. was an accident, which has brought about a very curious result which is now being watched. It is too early to report at the present time, however, on the success or non-success of this particular road.

Mr. Crosby.

The maintenance cost data shown in Tables 1 and 2 include taking care of the bleeding by putting on chips, taking care of the slipperiness, which was objectionable on some sections, and in one case it covers the removal of a very disagreeable mud which developed on the surface in the early spring following its construction.

TABLE 1.—MAINTENANCE COST DATA ON EIGHTEEN SECTIONS OF MARYLAND ROADS.

Section No.	Area, in square yards.	Material used.	Time when used.	Cost per square yard of resurfacing macadam.	Cost of pitching, including chipping.	Cost per square yard of maintenance, to 1/1/11.	Total cost per square yard of treatment, to 1/1/11.	Total cost per square yard of road surface, to 1/1/11.	Remarks.
1	11 226	Texaco.....	7 and 8/09	$0.337	$0.290	$0.040	$0.330	$0.666	
2	1 936	Gulf.......	8 and 9/09	0.339	0.432	0.130	0.561	0.900	
3	1 243	Texas......	9/09	0.336	0.759	0.160	0.920	1.256	
4	1 173	Fairfield...	9/09	0.337	0.449	0.161	0.610	0.947	
5	1 712	U. G. I.....	9/09	0.339	0.341	0.136	0.477	0.816	
6	1 908	Warren....	9 and 10/09	0.333	0.610	0.140	0.750	1.083	
7	1 909	Tarvia.....	10/09	0.337	0.618	0.107	0.725	1.062	
8	1 816	Tarite......	10/09	0.336	0.541	0.060	0.601	0.937	
9	1 264	U. G. I.....	11/09	0.340	0.451	0.092	0.543	0.883	
19	128	S. O........	11/09	0.241	0.162	0.085	0.247	0.488	Mat. gratis.
20	1 294	U. G. I.....	10/09	0.230	0.408	0.158	0.566	0.795	
21	694	Texas......	10/09	0.228	0.552	0.184	0.736	0.964	
22	977	Gulf........	10/09	0.229	0.411	0.272	0.682	0.911	
23	845	Warren....	10/09	0.224	0.691	0.031	0.722	0.946	
24	2 246	U. G. I.....	9/09	0.229	0.256	0.042	0.298	0.528	
25	1 532	Fairfield...	9/09	0.228	0.355	0.015	0.370	0.598	
26 and 27	5 207	Local and U. G. I.	8/09	0.216	0.262	0.016	0.278	0.494	

TABLE 2.—MAINTENANCE COST DATA ON EIGHT SECTIONS OF MARYLAND ROADS.

Section No.	Area, in square yards.	Material used.	Time when used.	Cost per square yard for resurfacing macadam.	Cost of pitching, including chipping, per square yard.	Total cost of road surfacing, per square yard.
10	3 532	U. G. I..............	May and June, 1910..........	$0.381	$0.292	$0.673
11	5 981	Texas..............	June, 1910..................	0.374	0.251	0.625
11-A	1 261	Mixed Tar..........	June, 1910..................	0.355	0.272	0.627
12	4 685	Headley............	June and July, 1910...... ...	0.384	0.174	0.558
13	4 203	B. A. P..............	July and August, 1910.......	0.373	0.400	0.773
14	3 184	Fairfield...........	August and September, 1910	0.387	0.269	0.656
15	5 808	Fairfield Anti-Dust.	October, 1910...............	0.385	0.069	0.454
16	1 100	U. G. I. Anti-Dust..	October and September, 1910	0.407	0.279	0.686

Mr. Crosby.

The traffic on this road has been counted as follows:

Section No. 1, City End.

One-horse vehicles	245	Average per day of 12 hours.
Two-horse vehicles	104	
Three-horse vehicles	10	
Four-horse vehicles	14	
Five-horse vehicles	..	
Six- (or more) horse vehicles	1	
Motor cycles	8	
Motor runabouts	26	
Motor touring cars (4 or 5 seats)	141	
Motor touring cars (6 or 7 seats)	66	
Motor wagons	4	

At times, these figures are greatly exceeded, such as in periods when there is racing at the course, the entrance of which is at the end of Section 3.

Sections Nos. 1 to 3, inclusive.

One-horse vehicles	195	Average per day of 12 hours.
Two-horse vehicles	101	
Three-horse vehicles	3	
Four-horse vehicles	19	
Six- (or more) horse vehicles	1	
Motor cycles	5	
Motor runabouts	15	
Motor touring cars (4 or 5 seats)	141	
Motor touring cars (6 or 7 seats)	55	
Motor wagons	7	

Sections Nos. 14 and 15.

One-horse vehicles	52	Average per day of 12 hours.
Two-horse vehicles	23	
Three-horse vehicles	3	
Four-horse vehicles	4	
Six-horse vehicles	1	
Motor cycles	7	
Motor runabouts	11	
Motor touring cars (4 or 5 seats)	94	
Motor touring cars (6 or 7 seats)	39	
Motor wagons	3	

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Sections Nos. 16 and 17.

One-horse vehicles	31	Average per day of 12 hours.
Two-horse vehicles	19	
Three-horse vehicles	3	
Four-horse vehicles	1	
Six-horse vehicles	1	
Motor cycles	3	
Motor runabouts	6	
Motor touring cars (4 or 5 seats)	69	
Motor touring cars (6 or 7 seats)	40	
Motor wagons	4	

Some other penetration work has been done, and some, such as on road surfaces of gravel and of oyster shells, is especially interesting. The speaker regrets his inability to report on this work at the present time.

Mr. Ross. CHARLES W. ROSS, ESQ.* (by letter).—In addition to a large amount of work by surface treatment, the penetration method has been used in Newton to some extent.

Several roadways were treated with preparations of 90% Heavy Standard Oil and with Tarite-Asphalt by the penetration method. The materials were heated sufficiently to make them flow freely through the sprinkler of the oil distributor. The macadam surface of the roadway was first spiked up and then covered with a 2-in. layer of screened stone. This stone came from an ordinary gravel pit, and varied in size from ½ to 2 in., the smaller stones filling in between the larger ones. The surface was rolled smooth and hard, after which the oil was applied hot. Used at the rate of about 1 gal. per sq. yd., the oil readily penetrated the entire mass. A coating of gravel which had passed through a ½-in. screen was then applied and thoroughly rolled with a steam roller. The road was opened to traffic within 12 hours after completion. The cost of work of this class varied from 21 to 23 cents per sq. yd., including all labor and materials.

Early in the spring of 1908 one of the principal streets was so badly out of shape that it required a new surface of about 3 in. of stone. It was decided to bind the new material with Tarvia A by the penetration process, and an area of 6 290 sq. yd. was surfaced at a cost of 46 cents per sq. yd. At about this time another street, having an area of 9 821 sq. yd., was treated in like manner with Tarvia A, except that a much thinner coating of stone was applied. The cost of this latter construction was 28½ cents per sq. yd.

*Street Commissioner, West Newton, Mass.

R. K. Compton, Esq.*—The following discussion will describe the methods of constructing bituminous macadam roads by the penetration method in Baltimore, Md. This experience extends over a period of 2½ years, but the following description pertains particularly to a stretch of road 1.55 miles in length, from 18 to 32 ft. in width, and having a total area of 17 788 sq. yd. The original roadbed was of ordinary macadam 6 in. thick. It had been in use about 4 years, and was badly worn in places. The entire stretch of road is subjected to heavy automobile traffic, and the portion west of the Stoney Run Bridge, particularly the north drive, is subjected to heavy team traffic from an adjacent sand and gravel plant. The team traffic, though slow moving, is poorly distributed, owing to the usual bad judgment exercised by drivers. Mr. Compton.

Where the original roadbed was in such condition as to require re-surfacing with new stone, the following method was used: The surface was first swept clean with the ordinary street-sweeping machine, then "spiked" with the roller, picked and raked by hand, and all dirt and foreign material removed. It was then rolled to a firm bed, and sufficient new stone was added to bring the surface to the proper cross-section. In no case was the thickness of new stone less than 3 in. after rolling, nor more than 6 in., so that the average thickness was about 4½ in. Where sufficient depth was available, a layer of stone in sizes from 2 to 1 in., known as No. 2, was first spread and then covered with stone dust in sufficient quantities to fill all interstices. Great care was used in this operation, as it was realized that if the bitumen penetrated the No. 2 course a slack roadbed would result. In order to secure the filling of all interstices, the dust was swept in, and water was used where necessary. Where water was used, however, this course was allowed to dry before placing the next, or No. 3 course.

After the No. 2 layer was completed a 3-in. layer of No. 3 stone, ranging in size from 1 to ¼ in. was laid. Where only the minimum depth of 3 in. was available, this size was used alone. The method of rolling was the same for all courses as with ordinary macadam, except that the last or No. 3 course was rolled firmly but not tightly, the idea being to allow the bitumen to penetrate not less than 2½ in.

After the macadam had been rolled, the binder was applied at the rate of not less than 1½ nor more than 2 gal. per sq. yd. of surface. Two materials were used; namely, "Fairfield Anti-Dust," a local material, which was applied at a temperature of not less than 190° Fahr., and "Standard Asphalt Binder" "B," which was applied at a temperature between 300° and 400° Fahr. An ordinary asphalt

* Assistant City Engineer, Baltimore, Md.

Mr. Compton. thermometer was used, and the binder was at all times applied at the temperature recommended by the manufacturer.

In heating the binder two kettles were used, one having a capacity of 200 gal., and the other a capacity of 400 gal. Both kettles were mounted on wheels, and were easily moved. The smaller kettle worked more satisfactorily with the "Fairfield Anti-Dust," which material runs freely from the barrel and is easily heated, while the larger kettle worked more satisfactorily with the "Standard," which does not run freely from the barrel and requires more heat, thereby requiring more storage capacity, if the work is to be handled economically. A 1½-in. hose, with a spreader on the extreme end, was attached to the rear of the kettle, the length of the hose being from 10 to 15 ft., depending on the width of the road. The binder was applied with this hose, and the flow was regulated by a valve on the rear of the kettle. A rubber hose was first tried, but it was found that it burned out too frequently under the extreme heat of the bitumen. It was then found that the ordinary linen hose, worth about 15 cents per ft., answered the purpose, and was, of course, much more economical. A more uniform distribution of the bitumen was obtained by using the hose than by hand-pouring. The former method is also more economical than hand-pouring, as the labor of heating and spreading bitumen, spreading Granolithic, and rolling it was 4 cents per sq. yd., as compared with 8 cents for the same items by hand-pouring. In all cases the bitumen was spread by beginning at the lower elevation of the roadbed and working up grade.

After the bitumen had been applied the entire surface was covered with Granolithic stone ranging in size from ⅝ to ¼ in., and free from dust. The roadbed was then thoroughly rolled. If the bitumen appeared on the surface, more Granolithic was added, and the rolling was continued until the material ceased to creep and became a solid mass, showing an even, hard, dense, and granular surface. The rolling was generally begun at the gutter lines, and carried toward the crown, the shoulders first being rolled to prevent spreading. For several weeks after the road was open to traffic it was noticed that the bitumen appeared on the surface in places. Granolithic was placed in convenient piles at the sides of the road, and was spread on these places when required. In some cases it was found necessary or advantageous to do more rolling.

Two rollers were used on the work, one weighing 12 and the other 6 tons. The 12-ton roller was used for "spiking," for rolling the No. 2 course, and at times for rolling the No. 3 course.

It was found, however, that the 6-ton roller worked very satisfactorily for the No. 3 course, and also for the final rolling on the Granolithic.

The cost of constructing 14 764 sq. yd. of roadway by this method was 72.95 cents per sq. yd., itemized as follows: Mr. Compton.

Labor	24.82	cents.
Stone	27.40	"
Rolling	4.84	"
Binder	13.74	"
Coal	0.95	"
Hauling binder	0.41	"
Incidentals, including supplies and repairs to plant	0.79	"
Total	72.95	cents.

The cost of rolling represents the hire of the 6-ton roller, at $10 per day, and the use of the Department's 12-ton roller, at $5 per day, which covers the salary of the engineer and watchman. The quantity of bituminous binder used was 1.9 gal. per sq. yd., which included a sealing coat on about 25% of the bituminous work. The bituminous binder flushed up very evenly on the greater part of the surface, so that the sealing coat was used only where it was deemed necessary.

The following rates of wages prevailed, per day of 8 hours:

Foreman	$3.50
Roller engineer	3.00
Labor	2.00
Carts	2.50
Double team	3.60
Watchman	2.00

The following prices were paid for materials:

Crushed stone, No. 2	$1.90	per cu. yd.
Crushed stone, No. 3	2.15	" " "
Crushed stone, dust	1.75	" " "
Crushed stone, Granolithic	2.15	" " "
Binder "Fairfield"	18.40	" ton of 2 000 lb.
Binder "Standard"	18.00	" " " 2 000 "
Coal	4.25	" "
Hauling binder	0.75	" "

"Gabbro" crushed stone was used throughout the entire work.

Except in a few places, the surface is in excellent shape at this time. The exceptions are in that portion of the work which was done during the latter part of September and the first part of October, when the nights and parts of the day were cool, thus preventing the bitumen from flushing to the surface in a satisfactory manner. This defect, it is believed, will correct itself in the spring when the warm

Mr. Compton. weather comes. The portion done during warmer weather has flushed up in a very satisfactory manner, and at present presents a bitulithic appearance. There are also at this time four or five weak places, each about 2 sq. ft. in area, due without doubt to the bitumen being too hot, which condition will occur at times at the bottom of the kettles.

Mr. Connell. WILLIAM H. CONNELL, ESQ.*—As a part of the experiments on White Plains Avenue, Borough of the Bronx, several sections were built by the penetration method, using both asphaltic and tar binders. The description and detailed costs of the sections follow:

WHITE PLAINS AVENUE EXPERIMENTAL PAVEMENTS.

SECTION TWELVE: 29 + 00 to 31 + 00.

Bituminous Pavement, Penetration Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and sufficient screenings to fill the voids. It was spread and rolled to a finished depth of 3½ in.

Surface Course or Paving Mixture.—Crushed trap rock (1½-in.) was spread and lightly rolled to a depth of 2¾ in. From 29 + 00 to 30 + 50 about 1½ gal. of Sanford and Strains Asphalt Binder was applied and a layer of ¾-in. stone was spread and rolled. From 30 + 50 to 31 + 00, the Standard Oil Company's Special Binder was used.

Seal Coat.—The quantity applied was ¾ gal. per sq. yd., over which coarse sand and chips were spread and rolled. An 18-ton roller was used.

COST PER SQUARE YARD.

Base course	$0.329
Surface course	0.501
Seal coat	0.123
Hauling binder	0.006
Tar and stone heaters	0.003
Fuel	0.021
	$0.983

DETAILED COST PER SQUARE YARD.

Base 3½ in. thick.

		Per square yard.
65.7 cu. yd. trap rock, at $1.65	$108.40	$0.229
13.1 " " screenings, " 1.65	21.68	0.046
Labor	......	0.054
Total		$0.329

SURFACE COURSE.

		Per square yard.
Labor	$93.56	$0.198
1⅛-in. stone, 45.9 cu. yd., at $1.65	75.73	0.161
¾-in. stone, 3.5 " " " 1.80	6.30	0.013
Binder, 719 gal., at $0.085	61.11	0.129 = 1.52 gal. per sq. yd.
472 sq. yd., at $0.501	$236.70	$0.501

* Assistant Commissioner of Public Works, Borough of the Bronx, New York City.

Mr. Connell.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	28	$4.00	$14.00
Pouring binder	1	16	2.25	4.50
Heating "	2	38	2.25	10.68
Carrying binder	3	34	2.25	9.56
Spreading top course 1½-in. stone	7	92	2.25	25.87
" ¾-in. stone	1	20	2.25	5.62
Wheeling stone	2	21	2.25	5.90
Heating stone	3	32	2.25	9.00
Rollerman	1	15	4.50	8.43
472 sq. yd., at $0.198				$93.56

SEAL COAT.

		Per square yard.
Labor	$21.31	$0.045
Chips, 3.5 cu. yd., at $1.90	6.65	0.014
Binder, 353 gal. " 0.085	30.01	0.064 = 0.75 gal. per sq. yd.
472 sq. yd. at $0.123	$57.97	$0.123

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	5	$4.00	$2.50
Pouring binder	1	5	2.25	1.40
Carrying "	3	12	2.25	3.37
Heating "	2	13	2.25	3.65
Spreading chips	1	7	2.25	1.96
Wheeling "	2	9	2.25	2.53
Heating "	2	11	2.25	3.09
Rollerman	1	5	4.50	2.81
				$21.31

The actual time required for laying the surface course (including the top course) was 3½ days, or an average of 135 sq. yd. per day. The ¾-in. stone required heating. Work was postponed three times on account of delayed shipments of binder, thereby increasing the cost. The actual time for laying the seal coat was 5 hours, or an average of 755 sq. yd. per day. The chips required heating.

SECTION ELEVEN: 27 + 97 to 29 + 00.

Barrett Manufacturing Company's "Modern Pavement."

Foundation Course.—This consists of 1½-in. crushed trap rock, spread and rolled to a finished depth of 3 in.

Surface Course or Paving Mixture.—A sufficient quantity of sand was spread to fill the voids. It was about ½ in. in thickness on top of the foundation course, over which about 1 gal. of Tarvia X per sq. yd. was applied. Then 1½-in. crushed trap rock was spread and

Mr. Connell. lightly rolled to a depth of 3 in. About 2 gal. of Tarvia X per sq. yd. was then applied by the penetration method. A layer of ¾-in. stone was spread and rolled. An 18-ton roller was used.

Seal Coat.—About 1 gal. per sq. yd. was applied, over which coarse sand was spread and rolled.

Cost per Square Yard.

Base course	$0.305
Surface course	0.698
Seal coat	0.219
Hauling binder	0.010
Tar heater	0.003
Fuel	0.021
	$1.256

Detailed Cost per Square Yard.

Base 3 in. thick.

		Per square yard.
33.8 cu. yd. trap rock, at $1.65	$55.77	$0.229
3.3 " " screenings, at 1.65	5.57	0.022
Labor	......	0.054
		$0.305

Surface Course.

		Per square yard.	
Labor	$59.06	$0.243	
1½-in. stone, 19.6 cu. yd., at $1.65	32.34	0.133	
Sand, 6.5 " " " 1.21	7.86	0.032	
Binder 1st coat, 262 gal., " 0.095	24.89	0.102	= 1.08 gal. per sq. yd
Binder 2d coat, 480 gal., " 0.095	45.60	0.188	= 1.97 gal. per sq. yd.
243 sq. yd., at $0.698	$169.75	$0.698	

Field Labor.

	No.	Hours.	Rate.	First Coat.
Foreman	1	6	$4.00	$3.00
Pouring	3	12	2.25	3.37
Heating binder	2	13	2.25	3.65
Spreading sand	10	20	2.25	5.62
Rollerman	1	2	4.50	1.12
243 sq. yd., at $0.069				$16.76
				Second Coat.
Foreman	1	16	$4.00	$8.00
Pouring	3	9	2.25	2.53
	2	16	2.25	4.50
Heating binder	3	9	2.25	2.53
	2	16	2.25	4.50
Spreading 1½-in. stone	10	20	2.25	5.62
	10	30	2.25	8.43
Rollerman	1	11	4.50	6.19
243 sq. yd., at $0.174				$42.30

Mr. Connell.

Seal Coat.

					Per square yard.	
Labor				$17.32	$0.071	
¾-in. stone,	5.0 cu. yd.,	at	$1.80	9.00	0.037	
Sand,	2.0 " "	"	1.21	2.42	0.010	
Screenings,	1.0 " "	"	1.65	1.65	0.007	
Binder,	240 gal.	"	0.095	22.80	0.094	= 0.99 gal. per sq. yd.
243 sq. yd., at $0.219				$53.19	$0.219	

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	6	$4.00	$3.00
Heating binder	1	6	2.25	1.68
Pouring binder	2	10	2.25	2.81
Spreading sand	2	10	2.25	2.81
Spreading ¾-in. stone	2	16	2.25	4.50
Spreading screenings	3	3	2.25	0.84
Rollerman	1	3	4.50	1.68
				$17.32

The actual time for laying the surface course was: 1st coat, 6 hours, or an average of 324 sq. yd. per day; 2d coat, 8 hours, or an average of 243 sq. yd. per day.

The actual time for laying the seal coat was 6 hours, or an average of 324 sq. yd. per day.

Section Fifteen: 34 + 50 to 36 + 25.

Bituminous Pavement, Penetration Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3½ in.

Surface Course or Paving Mixture.—Crushed trap rock (1½-in.) was spread and lightly rolled to a depth of 2½ in. Tarvia—2.3 gal. per sq. yd.—was then applied, after which a layer of chips was spread and rolled.

Seal Coat.—About ¾ gal. of Tarvia per sq. yd. was applied, over which chips were spread and rolled with an 18-ton roller.

Cost per Square Yard.

Base course	$0.305
Surface course and seal coat	0.735
Fuel	0.021
Hauling binder	0.008
Tar heater	0.002
	$1.071

Detailed Cost per Square Yard.

Base 3½ in. thick.

		Per square yard.
57.4 cu. yd. trap rock, at $1.65	$94.71	$0.229
5.7 " " screenings, " 1.65	9.47	0.022
Labor		0.054
		$0.305

Mr. Connell.

SURFACE COURSE AND SEAL COAT COMBINED.

		Per square yard.
Labor	$100.77	$0.244
1½-in. stone, 40.2 cu. yd., at $1.65	66.33	0.161
Chips, 80 cu. yd., at $1.65	13.20	0.032
Binder, 1 296 gal., at $0.095	123.12	0.298 = 3.14 gal. per sq. yd.
413 sq. yd., at $0.735	$303.42	$0.735

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	48	$4.00	$24.00
Pouring and carrying tar	3	106	2.25	29.81
Heating tar	1	11	2.25	3.08
	2	48	2.25	13.50
Spreading 1½-in. stone	15	60	2.25	16.88
Spreading chips	1	16	2.25	4.50
Rollerman	1	16	4.50	9.00
				$100.77

The costs of the first coat of binder and the seal coat were not separated. About 0.75 gal. of binder per sq. yd. was used for a flush coat.

The actual time of laying was 5 days (4 full days and 2 part days), or an average of 80 sq. yd. per day. The length of time was due to poor supervision.

SECTION SIXTEEN: 36 + 25 to 37 + 75.

Bituminous Pavement, Penetration Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3½ in.

Surface Course or Paving Mixture.—Crushed trap rock (1½-in.) was spread and slightly rolled to a depth of 3 in., then 1½ gal. of Bermudez Road Asphalt per sq. yd. were applied, after which a layer of chips was spread and rolled.

Seal Coat.—Bermudez (½ gal. per sq. yd.) was applied, over which chips were spread and rolled, an 18-ton roller being used.

COST PER SQUARE YARD.

Base course	$0.305
Surface course	0.519
Seal coat	0.123
Hauling binder	0.005
Tar heaters	0.002
Fuel	0.021
	$0.975

Mr. Connell.

Detailed Cost per Square Yard.

Base 3½ in. thick.

		Per square yard.
49.2 cu. yd. trap rock, at $1.65	$81.18	$0.229
4.9 " " screenings, at $1.65	8.12	0.022
Labor		0.054
		$0.305

Surface Course.

		Per square yard.
Labor	$45.14	$0.128
1⅛-in. stone, 34.5 cu. yd., at $1.65	56.92	0.161
Chips........ 5.0 " " " 1.90	9.50	0.027
Binder....... 550 gal., at 0.131	72.05	0.203 = 1.55 gal. per sq. yd.
354 sq. yd., at $0.519	$183.61	$0.519

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	11	$4.00	$5.50
Pouring asphalt	2	18	2.25	5.06
Heating "	2	19	2.25	5.34
Carrying "	2	18	2.25	5.06
Spreading 1⅛-in. stone chips	5	36	2.25	10.12
	2	18	2.25	5.06
Rollerman	1	16	4.50	9.00
				$45.14

Seal Coat.

		Per square yard.
Labor	$11.53	$0.032
Chips, 5.0 cu. yd., at $1.90	9.50	0.027
Binder, 173 gal., at $0.131	22.66	0.064 = 0.49 gal. per sq. yd.
354 sq. yd., at $0.123	$43.69	$0.123

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	4	$4.00	$2.00
Pouring asphalt	2	6	2.25	1.68
Squeegee	1	3	2.25	0.84
Heating asphalt	1	3	2.25	0.84
Carrying "	2	6	2.25	1.68
Spreading chips	2	6	2.25	1.68
	4	4	2.25	1.12
Rollerman	1	3	4.50	1.69
				$11.53

Nothing except the binder was heated on the ground. The chips were delivered hot from the plant, and remained warm until the end of the job.

Mr. Connell. The actual time of laying the surface course was 1 day and 3 hours, or an average of 257 sq. yd. per day. The actual time of laying the seal coat was 4 hours, or an average of 708 sq. yd. per day.

SECTION SEVENTEEN: 37 + 75 to 39 + 22.

Bituminous Pavement, Penetration Method.

Surface and Base Course Spread in One Course.—This consists of 1½-in. crushed trap rock, spread and rolled to a depth of 6 in. Standard Oil Company's Binder "B" (1⅓ gal. per sq. yd.) was then applied, over which a layer of ¾-in. stone was spread and rolled.

Seal Coat.—Binder "B" (1.1 gal. per sq. yd.) was applied, after which chips were spread and rolled, an 18-ton roller being used.

COST PER SQUARE YARD.

Surface and base course	$0.648
Seal coat	0.177
Fuel	0.021
Hauling binder	0.006
Tar heater	0.001
	$0.853

DETAILED COST PER SQUARE YARD.

Surface and Base Course.

		Per square yard.
Labor	$41.90	$0.121
1½-in. stone, 82.0 cu. yd., at $1.65.	135.30	0.390
¾-in. stone, 5.0 " " " 1.80.	9.00	0.026
Binder, 480 gal., at 0.08.	38.40	0.111 = 1.38 gal. per sq. yd.
347 sq. yd., at $0.648	$224.60	$0.648

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	18	$4.00	$9.00
Pouring and carrying oil	3	48	2.25	13.50
Heating oil	2	25	2.25	7.03
Spreading 1½-in. stone	10	20	2.25	5.62
" ¾ " "	2	16	2.25	4.50
Rollerman	1	4	4.50	2.25
347 sq. yd., at $0.121				$41.90

SEAL COAT.

		Per square yard.
Labor	$25.12	$0.072
Chips, 3.0 cu. yd., at $1.90	5.70	0.017
Binder, 384 gal., at 0.08	30.72	0.088 = 1.11 gal. per sq. yd,
347 sq. yd., at $0.177	$61.54	$0.177

LABOR. Mr. Connell.

	No.	Hours.	Rate.	Totals.
Foreman	1	12	$4.00	$6.00
Pouring and carrying oil	2	24	2.25	6.75
Heating oil	1	13	2.25	3.65
Spreading chips	2	23	2.25	6.47
Rollerman	1	4	4.50	2.25
				$25.12

The actual time of laying the surface course, excluding the base course, was 2¼ days, or an average of 154 sq. yd. per day. The actual time of laying the seal coat was 1½ days, or an average of 231 sq. yd. per day. Nothing but the binder was heated. The 1½-in. trap rock was spread in one course, without screenings.

TEMPERATURE, WEATHER, ETC.

Pavement.	Date.		Temperature. 9 A. M.	12 NOON.	3 P. M.	Weather.
Standard Oil Penetration	Oct. 18, '10.	Begun.	59	65	66	Clear.
	Oct. 19, '10.		59	65	68	Clear.
	Oct. 21, '10.		55	57	59	Cloudy.
	Oct. 24, '10.		52	54	56	Clear.
	Oct. 25, '10.	Finished.	55	60	60	Cloudy.
Tarvia Penetration	Oct. 13, '10.	Begun.	48	52	55	Clear.
	Oct. 14, '10.		60	68	69	Fair.
	Oct. 15, '10.		61	63	..	Cloudy.
	Oct. 17, '10.		60	65	66	Cloudy.
	Oct. 18, '10.		59	65	66	Clear.
	Oct. 19, '10.	Finished.	59	65	68	Clear.
Barber Penetration	Oct. 26, '10.	Begun.	49	54	56	Clear.
	Oct. 27, '10.		58	65	67	Cloudy.
	Oct. 28, '10.	Finished.	54	55	56	Clear.
"Modern Pavement"	Oct. 19, '10.	Begun.	59	65	68	Clear.
	Oct. 21, '10.		55	57	59	Cloudy.
	Oct. 22, '10.		52	53	..	Rain.
	Oct. 24, '10.		52	54	56	Clear.
	Oct. 25, '10.	Finished.	55	60	60	Cloudy.
Sanford and Strains (Penetration)	Nov. 14, '10.	Begun.	38	44	43	Cloudy.
	Nov. 15, '10.		33	39	40	Cloudy.
	Nov. 17, '10.		38	40	42	Clear.
	Nov. 18, '10.		38	44	40	Clear.
	Nov. 22, '10.	Finished.	37	42	46	Overcast.

THE USE OF BITUMINOUS MATERIALS BY MIXING METHODS.

Mr. Blanchard.

ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Bituminous pavements constructed by mixing methods may be considered under two types. Type A has as its characteristic a carefully-graded aggregate, and may be classified with sheet-asphalt and wood-block pavements, from the standpoint of adaptability under certain traffic conditions, while Type B, consisting of a pavement with an upper course containing a larger percentage of voids than Type A, is suitable for residential streets in cities, roads on State trunk lines, etc., which are subjected to excessive motor-car traffic and heavy commercial traffic, considered from the standpoint of the present traffic on trunk highways.

In the construction of bituminous pavements of Type A, the materials for the aggregate are broken stone and sand of graded sizes mixed so as to form a dense mixture, while the usual bituminous materials are natural asphalts, oil-asphalts, and refined tars, either alone or in combination with asphaltic materials. In connection with the construction, machines are used to heat, and, in some cases, to clean the aggregate, and thoroughly mix it with the bituminous binders. These plants are of two kinds, stationary and portable. The results secured by the use of pavements of this character have been very satisfactory, from the standpoint of durability, but in some instances the surfaces have been slippery and in other cases dusty. Particular attention should be given in the future to the nature of the material used as a flush coat, as it is possible to construct a pavement of this type so that the surface will not be either slippery or dusty.

Pavements of Type B are usually constructed by using as an aggregate either the run of the crusher or a mixture of certain sizes of broken stone, with or without sand, and a bituminous cement of natural asphalt, oil-asphalt, heavy asphaltic oil, tar, or a combination of tar and asphaltic materials. The two methods which have been adopted in constructing pavements of this type are, first, the hand-mixing method, in connection with which unheated stone is generally used, and second, by mixing the aggregate and the bituminous material with mechanical mixers. The results—to date—by these methods in some cases have proved satisfactory and in others both insufficient and uneconomical. The results have generally proved satisfactory when the method and the materials were adaptable to local conditions, when the roads were well constructed with a bituminous cement of good quality, and the work was in charge of a competent inspector. Here, again, the function of the flush coat and the course of mixed aggregate has not been fully appreciated. In the future much attention should be devoted to the design of a mixer, including a dryer as an essential adjunct under certain conditions. One type of plant of this character should be designed so as to be economically adaptable to road work on which portable crushers are used, the output of which is necessarily limited.

Distributors for the application of a flush coat should also receive attention. Mr. Blanchard.

In Rhode Island bituminous pavements have been constructed by the mixing method since 1906. Judging from the results obtained to date the most economical and efficient pavements for heavy combination trunk line traffic under normal conditions in Rhode Island have been those constructed with tar in the mix and an oil-asphalt flush coat, the oil-asphalt which has proved most satisfactory during all seasons of the year being a Texaco asphalt having a melting point of from 90° to 95° cent., as determined by the test proposed by the Special Committee of this Society. The first pavement of this type was built in 1908 in East Providence.* When inspected, in January, 1911, the surface was in an excellent state of preservation and practically dustless, the flush coat of asphalt being in such condition as to take the prints of the calks of horses' hoofs, although the air temperature was 0° cent. The average traffic for 12 hours on this road during the summer of 1909 was 355 horse-drawn vehicles and 810 motor vehicles. Under normal conditions, when constructed under competent supervision, and in the months of June to September, inclusive, a pavement of the class referred to will cost from 25 to 35 cents per sq. yd. more than the cost of ordinary macadam.

For roads subjected to traffic consisting of 90% motor cars and a light horse-drawn vehicle traffic, the results in Rhode Island indicate that a flush coat on a bituminous pavement constructed by the mixing method is not necessary. The excellent condition, when examined in October, 1910, of the 1906 section in Charlestown built of Providence tar, and the 1908 section in Narragansett built of Tarvia, are evidence that a road of this type is economical and efficacious. The summer traffic during 1909 on the Narragansett road averaged 42 horse-drawn vehicles and 410 motor vehicles for a period of 12 hours. A road of this character can be constructed, under normal conditions, during the months from June to September, inclusive, with refined tar or an oil-asphalt for from 15 to 25 cents per sq. yd. in excess of the cost of ordinary macadam, provided the construction work is supervised by a competent inspector. The estimates given are based on 1910 labor and material cost data.

The 1909 experimental sections built in Barrington, while the speaker was in charge of bituminous work in Rhode Island, have been open to traffic for more than a year. The sections were all built by the mixing method, each section having a paint coat of the same kind of bituminous material as was used in the mix. The materials were Malden Tarite, Springfield Tarite, Tarite-Asphalt E (20% asphalt), Tarite-Asphalt J (10% asphalt), Texaco Macadam Binder and the U. G. I. Road Compound No. 4. When examined in January, 1911, the various

*The construction of this section of the East Providence road is described in *Transactions*, Am. Soc. C. E., Vol. LXI, p. 445 *et seq.*

Mr. Blanchard.

sections exhibited the following characteristics: The Malden Tarite and Springfield Tarite sections presented an excellent, close-grained, mosaic surface, the surface coat on the top of the stones having entirely disappeared. The surface afforded an excellent foothold for horses. There has been a slight abrasion of the road metal composing the upper surface. Very little difference was noted between the Tarite-Asphalt E and Tarite-Asphalt J sections, except that the mosaic surface of the stones was more apparent in the latter than in the former. The surface presented a fairly good foothold for horses, and was practically dustless. On the section constructed with Texaco Macadam Binder the stones composing the upper layer were not visible. The flush coat was remarkably dustless, and, at a temperature of 0° cent., was slightly impressed by the calks of horses' hoofs. On the surface constructed with the U. G. I. Road Compound No. 4 the stones were not visible. The surface was extremely hard and smooth, and hence slippery, but was dustless.

The road on which these various sections were constructed is between West Barrington and Nayatt Point, in the Town of Barrington. Each section was built on a tangent, and the grades varied from 0.50 to 1.14 per cent. The sub-grade encountered was composed of a very hard layer of clay and sand over a foundation bed of bricks, which had been placed there several years before by the town highway authorities. Almost the entire length of the road was through a wooded district, the shade being very dense. Previous to reconstruction, the road had been surfaced with gravel. In June and July, 1909, the average traffic for 8 hours was as follows:

Horse-drawn vehicle traffic.	One-horse vehicles	71
	Two-horse vehicles	3
Motor vehicle traffic.	Motor cycles	1
	Motor runabouts	9
	Motor touring cars (four or five seats)	43
	Motor touring cars (six or seven seats) including limousines and landaulets	17

The contract was let originally for ordinary macadam construction, and later a secondary agreement was drawn up by which the extra labor required on the bituminous work was paid for as per agreement covering prices for different grades of labor plus a commission of 15 per cent. The prices paid to laborers on bituminous work were $2.00 per day of 10 hours for experienced men, and $1.75 per day for ordinary laborers. One-horse teams and drivers were paid $3.50, and two-horse teams and drivers $5.00 per day.

The following plant equipment was used: A Kelly-Springfield 15-ton roller; a tar-coating machine manufactured by the American

Tar Company, the rent of which was $2.00 per day. The contractor supplied three tar kettles, mixing platforms, square-pointed shovels, pails, dippers, and rakes, at a rent of $0.25 per kettle and accessories per day. Brooms, potato-hooks, and steel-bodied wheel-barrows were supplied by the State. Mr. Blanchard.

The macadam was built in two courses: The foundation course consisted of broken stone varying from 1½ to 2¾ in. in longest dimensions, which was 6 in. deep when loose, and was rolled to 4 in. The top course consisted of stone of the same quality, varying in size from ¾ to 1½ in., which was 3 in. deep when loose, and was rolled to 2 in. The rock used was a chlorite gneiss, having the following characteristics:

Determinations.	Results.
Specific gravity	2.80
Weight per cubic foot	175 lb.
Water absorbed per cubic foot	0.46 lb.
Percentage of wear	4.6
French coefficient of wear	8.6
Hardness	15.7
Toughness	8
Cementing value	Fair.

The width of macadam was 14 ft. and the crown throughout was ¾ in. per ft. The top-course stone was mixed by hand with bituminous material until all stones were thoroughly coated. Most of the mixing was done by hand, with long-handled, square-pointed shovels, on plank platforms composed of three sections, each 8 ft. square. The tar-coating machine was used on the sections built with Springfield-Tarite and the U. G. I. No. 4 Road Compound. This machine and the method of using it was described by the speaker in a paper on "Use of Binding Materials in the Construction of Metalled Roads."* The mixing gang consisted of three experienced and five common laborers. The flush-coat gang consisted of one experienced and two common laborers. In applying the flush-coat house brooms were used, the material being supplied to the spreader in steel-bodied wheel-barrows.

Table 1 contains general information and Table 2 cost data relative to the various experimental sections; Table 3 gives analyses of the bituminous materials used. The analyses were made in accordance with the methods proposed by the Special Committee of this Society appointed "To Report on Bituminous Materials for Road Construction, and on Standards for their Test and Use."

The following notes, relative to details of construction are of interest; they were taken from the records of the Resident Engineer, Irving W. Patterson, Jun. Am. Soc. C. E.

* Presented before the Second International Road Congress at Brussels, in August, 1910.

Mr. Blanchard.

TABLE 1.—GENERAL INFORMATION.

Material.	Length, in feet.	Average grade, percentage.	Air Temperature, in Degrees, Fahrenheit.		Date mixing began.	Date mixing ended.	Average temperature of material, in degrees, Fahrenheit.	Bituminous material in mix, in gallons per square yard.	Bituminous material in flush coat, in gallons per square yard.	Daily progress, in square yards (mix).	Daily progress, in square yards (paint).
			Average.	Min.							
Malden Tarite	350	0.50	80	60	Sept. 20.	Sept. 22.	240	$1\frac{5}{8}$	$\frac{5}{8}$	232	550
Texaco Macadam Binder	280	2.12	80	65	Sept. 22.	Sept. 29.	255	$1\frac{3}{4}$	$\frac{5}{8}$	217	400
Tarite-Asphalt (20%)	168	0.28	70	55	Sept. 29.	Sept. 30.	255	2	$\frac{5}{8}$	216	400
Tarite-Asphalt (10%)	169	0.83	65	50	Sept. 30.	Oct. 2.	250	2	$\frac{5}{8}$	200	400
Springfield Tarite	291	1.14	75	50	Oct. 2.	Oct. 6.	240	2	$\frac{5}{8}$	150	500
U. G. I. Road Compound No. 4.	275	1.14	75	50	Oct. 6.	Oct. 9.	290	2	$\frac{5}{8}$	155	450

TABLE 2.—COST DATA.

Material.	Material per gallon, f. o. b.	Freight, per gallon.	Loading and hauling, per gallon.	Material on road, per gallon.	Heating material, per gallon.	Mixing material, per gallon.	Painting, per gallon.	Total cost of material in place, per square yard.*
Malden Tarite	$0.080	$0.010	$0.007	$0.097	$0.008	$0.029	$0.021	$0.297
Texaco Macadam Binder	0.110	Price per gallon includes freight.	0.005	0.115	0.008	0.030	0.021	0.357
Tarite-Asphalt (20%)	0.098	0.009	0.004	0.111	0.008	0.026	0.033	0.383
Tarite-Asphalt (10%)	0.092	0.009	0.004	0.106	0.008	0.028	0.036	0.376
Springfield Tarite	0.080	0.009	0.004	0.093	0.007	0.043	0.032	0.369
U. G. I. Road Compound No. 4.	0.070	0.025	0.003	0.099	0.007	0.044	0.037	0.391

* The total cost per square yard given in Table 2 is the cost over and above the cost of ordinary macadam. The cost of a 2-in. course of ordinary macadam was 16 cents per sq. yd.

TABLE 3.—ANALYSES OF BITUMINOUS MATERIALS.

Mr. Blanchard.

	Malden Tarite.	Texaco Macadam Binder.	Tarite-Asphalt (20%).	Tarite-Asphalt (10%).	Spring-field Tarite.	U. G. I. Road Com-pound No. 4.
Specific gravity	1.222	0.965	1.176	1.211	1.244	1.167
Water soluble, organic	0.4	0.3	0.5	0.6	0.2	0.3
Water soluble, inorganic	0.4	0.0	0.1	0.0	0.4	0.1
Free carbon	25.0	0.3	18.6	22.5	24.5	2.4
Ash	0.1	0.0	0.0	0.1	0.0	0.1
Solubility in cold $C\,Cl_3$	....	99.6	74.0	69.5	....	....
Fixed carbon	29.4	8.0	27.4	26.2	30.7	22.1
Paraffine	....	0.2	....	....	....	....
Melting point, original material	....	....	....	....	....	....
Evaporation in 5 hours at 170° cent	15.0	16.6	10.5	13.5	11.9	15.5
Melting point residue	62	65	70	67	63	74
Penetration residue, 4° cent	2	46	2	1	3	0.5
Penetration residue, 25° cent	15	105	12	10	15	3
Evaporation in 5 hours at 205° cent	....	19.1	17.5	18.0	....	....
Melting point residue	....	116	80	85	....	....
Penetration residue, 4° cent	....	29	1	1	....	....
Penetration residue, 25° cent	....	58	2	2	....	....
Solubility 88° Baumé naphtha	....	77	....	....	....	....
Character of solution, sticky or oily	....	sticky	....	....	....	....
Distillation to 300° cent	19.7	....	....	18.7	20.7	16.8
Up to 105° cent	0.0	....	....	0.3	0.0	0.0
105° to 170° cent	0.4	....	....	1.0	0.3	0.0
170° to 225° cent	7.8	....	....	3.3	4.8	0.0
225° to 270° cent	8.3	....	....	9.4	10.9	10.4
270° to 300° cent	3.2	....	....	4.7	4.7	6.4

Malden Tarite.—At the prevailing temperature, the material refused to run from the bungs of the barrels with sufficient rapidity to keep the work constantly progressing. For this reason the heads of the barrels were knocked out and the contents emptied into the kettles in a very short time. On this section rolling was done each day until a thorough compacting had been effected and the material had set up so that there was no appreciable movement under the wheels of the roller. Rolling was never done until the material had been down over night. The first day rolling was done about 9 A. M., before the material had softened excessively. Later, as the material began to set up, rolling was delayed until the middle of the day because in the earlier hours the stiffness of the surface did not allow of compacting. The surface had set up in such a manner that painting might have been done at the end of about 6 days.

Texaco Macadam Binder.—The weather conditions during the construction of this section were the most unfavorable during the entire period of construction. Owing to frequent rains and almost constant mists, it was impossible to keep the stone dry. Work was carried on, however, except when rain was actually falling, because of the lateness of the season and the desire to finish to a certain point before weather conditions prevented further bituminous construction. The

Mr. Blanchard.

stone was mixed with no great difficulty when dry, but when damp, as was generally the case, it was very difficult to get it coated. The material was exceedingly slow in setting up. It was impossible to apply the flush coat until 3 weeks after laying the stone. Even at that time the surface felt very elastic under foot pressure.

Tarite-Asphalt (10%).—The heads of the barrels had to be removed in order to allow the material to flow into the kettles. Mixing was difficult, because of the low temperature and the consequent stiffness of the bituminous materials. Owing to the rapidity with which the material hardened on contact with the cold stones, it was necessary to mix very small patches at one time. Rolling on this section was done in the middle of the day. Rolling was also done on the day the material was laid because if the surface was left unrolled over night permanent compacting was made very difficult. At the end of 2 days the surface had set up sufficiently to allow painting.

Tarite-Asphalt (20%).—The quantity of material used, the manner of rolling, and the length of time necessary for setting up varied very slightly from the experience with Tarite-Asphalt (10%), but there was somewhat more difficulty in removing the material from the barrels.

Springfield Tarite.—Rolling was done on this section in the warm part of the day, because the season was rather late, and the mornings and evenings being cold, the material was stiffened so much that rolling did not compact the surface well. The flush-coat might have been applied on the third day.

U. G. I. No. 4 Road Compound.—The first section laid with this material was allowed to remain over night before rolling, but this section was never compacted as well as where the rolling followed close after the laying of the surface. Rolling in the early morning appeared to affect the surface in no way except to break up the stones of the No. 2 course. Rolling in the middle of the day, except on the day on which the material was laid, appeared to be but slightly more effective than rolling in the early morning. Hence it was found absolutely necessary to roll very soon after laying the No. 2 course. The great change produced by relatively slight changes of temperature can be judged from the fact that in the early morning at this season of the year the material in the barrels might be split to pieces with an axe, while earlier in the season it would flow slowly from the bungs.

Based on experience acquired while in charge of bituminous work in Rhode Island, and investigations conducted in the United States and Europe, the speaker considers that many of the failures of bituminous pavements constructed by mixing methods have been due to one or more of the following factors: Poor bituminous binder; bituminous material not adapted to the method used; the use of an insufficient quantity of bituminous material; poor mix; an aggregate unsatisfactory in quality; injudicious selection of sizes of road metal in the aggre-

gate; aggregate covered with dust or dirt; damp broken stone or sand; overheated broken stone, sand, or bituminous material; the use of excessively heavy three-wheel rollers; the coating of the bituminous-covered aggregate with roadside dust, either before rolling or preceding the application of the flush-coat; insufficient bond between the foundation course of macadam and the bituminous course, due in certain cases to over-compacting the lower course; and, finally, incompetent supervision of the handling of materials and of the details of construction. Mr. Blanchard.

F. C. Pillsbury, M. Am. Soc. C. E.—Road surfaces should be designed, as nearly as possible, for the traffic to be sustained, and, at the present time, it seems that it is possible to draw the line closely between the necessity for a mixed bituminous macadam and for a less expensive type on the one hand, and a superior pavement on the other. Mr. Pillsbury.

There is no doubt that it is possible to construct on a suitable base a mixed bituminous macadam which will sustain extremely heavy horse-drawn vehicle traffic, and it is even possible that in the future a pavement may be found which will sustain the very heaviest horse-drawn vehicle traffic. On the other hand, there is no doubt that even the most successful bituminous macadam constructed by penetration or layer methods will not sustain a traffic equal in weight to that which may be borne by the mixed composition.

This seems to prove the superiority of a mixed aggregate over other kinds of bituminous macadam when heavy loads drawn by horses must be sustained. For medium traffic, partly or even almost altogether automobile, it seems that the penetration and layer methods of construction will answer. They are less expensive, of course, but if the mixing method is superior in one case, why should it not be in the other? It is probable that its superiority will give much greater permanence, and that this may prove it to be more economical. On the other hand, a large volume of swiftly-moving automobile traffic may disturb the actual surface, causing abrasion to such an extent that either mixed, penetration, or layer treatment will be badly disturbed on the surface after two or three years, so that it would be economy in any case to restore the surface continually before destruction by surface applications of some form of bitumen.

The speaker does not intend these remarks to be an argument in favor of one method of construction over another; they are simply made for the purpose of calling attention to the fact that it is of first importance to consider whether it is economy to construct bituminous macadam by the mixing method instead of laying the most expensive pavements, such as vitrified brick, wood block, sheet-asphalt, first-class granite, etc., or instead of cheaper types of bituminous surfaces.

The speaker believes that there are many cases where an asphaltic bituminous macadam could be laid which would answer all the pur-

Mr. Pillsbury. poses now served by brick or wood block, except that wood block is quieter. Bituminous macadam, however, is quieter than brick or granite-block pavements. The cost of such pavement should not exceed one-half that of the brick or other more expensive forms mentioned above. Asphaltic bituminous macadam must be constructed only of selected materials placed with the greatest care.

The actual operation of mixing and placing an aggregate may be accomplished at a low cost by careless methods, but the results will be uncertain, and, without doubt, will result in failures in many cases. So much depends on the personal factor, that skilled and experienced labor should be employed. This leads to a question of machinery *versus* hand labor. From experience the speaker has no doubt that machine work is more satisfactory, as machinery may be used which will mix uniformly and thoroughly. It is almost impossible to obtain a uniform product by hand mixing.

One of the most interesting roads, on which both hand and machine mixing were used, was constructed in the summer of 1910 on the State highway between the City of Lynn and the Town of Revere, in Massachusetts. This road carries all the business traffic between Boston and Lynn and other cities and towns north and east, as well as a large volume of automobile travel.

If the road had not been subject to very heavy horse-drawn vehicle traffic, a mixed bituminous macadam would not have been constructed. The traffic was so heavy that the Massachusetts Highway Commission thought it advisable to use a bituminous material which was, as far as known, least likely to oxidize and lose its plastic and adhesive qualities until after the greatest period of wear had been obtained. It was finally decided to use a composition of natural oil and refined natural asphalt, and to mix this in a thorough manner with an excellent quality of trap rock.

The work was complicated and made difficult and expensive by the necessity of keeping the road open to travel. The entire width of the roadway was only 22 ft., hence only 11 ft. were available at one time.

At first the mixing was done by hand, but the cool nights and mornings made it difficult to combine the stiff bituminous material with the cold stone satisfactorily, and it was found that the work was progressing too slowly. Hence, it was decided to mix by machinery. Two Smith concrete mixers were used. The stone was heated and the mixing accomplished with entire satisfaction. The two Smith mixers turned out daily sufficient material to cover about 800 sq. yd. of surface. This amount kept three steam rollers busy, and was all that could possibly be handled.

This practical type of surface and the method of operation is to be strongly recommended for roads which are somewhat remote from a central mixing plant and are subjected to heavy traffic.

FRANK J. EPPELE, ESQ.* (by letter).—In Mercer County, New Jersey, four pieces of road have been re-surfaced to a depth of 1½ in. or more after compression with materials called by some special trade name, where the mineral aggregate was coated mechanically with a bituminous binder. These roads are giving by far the best and most satisfactory results. Mr. Eppele.

This work has been done at a cost of about 70 cents per sq. yd., under a guaranty of 5 years, and, after 2 years of service, does not show any appreciable indication of wear; in fact, judging by present surface conditions, these roads should last for a period of 15 years without any material expense. The writer understands, however, that similar work would cost at the present time from 80 to 90 cents per sq. yd., due to increases in the cost of labor and materials, and the question of royalties to be paid under patents granted for the construction of a bituminous road having 21% or less of voids.

A mineral aggregate of the desired proportions as to size, coated mechanically with the required amount of a first-class bitumen of the proper consistency, will, without doubt, give an ideal wearing surface for new as well as re-surfaced roads, but, under existing circumstances, and in accord with a recent legal decision, it is practically impossible to compile a specification, under which bids could be received with unlimited competition, without infringement on certain patent rights. On the other hand, to ask for bids under a specification calling for a restricted form of construction, or any special kind of pavement, would immediately meet with disfavor in the eyes of the public, and raise the cry of favoritism, regardless of the merits involved. Without open competition, it is useless to expect the cost of this form of construction to be brought within the limit of reasonable expenditure for general highway improvement, and so its use will probably be restricted to districts of fixed realty value and population. If this is true, we are forced to eliminate, when considering methods of bituminous construction, the mechanical coating of the mineral aggregate having a low percentage of voids, and to confine our thoughts practically to the spreading of the road metal, the application of the asphaltic binder to the surface, by hand or otherwise, and then to depend on its penetration qualities to give a variable condition of coated work.

Being dependent, then, on the penetration method to produce the form of construction for future highway improvement which will be within an allowable range of cost, as well as a method of re-surfacing roads already constructed, the main question seems to be one of proper asphaltic binder to be used. On this question it is the writer's judgment that a fluxed natural asphalt will give the best and most lasting results, and, in the end, prove to be the cheapest bitumen to use, even

*County Engineer, Mercer County, New Jersey.

Mr. Eppele. if the initial cost is somewhat above that of the artificial bitumens now on the market.

Mr. Connell. WILLIAM H. CONNELL, ESQ.*—The following discussion is confined to a general description of the experimental sections on White Plains Road which were built in order that a comparison might be made as to the cost and durability of bituminous pavements laid by the mixing and by penetration methods, with water-bound macadam and other types of pavements.

The White Plains Road experimental pavements consist of eighteen sections, ranging in length from 100 to 350 ft., with a width of 21 ft. Fourteen of these sections comprise the bituminous pavement group, two of which consist of a wearing surface of broken stone and sand with an asphaltic binder, built by the mixing method and laid on a concrete foundation. The wearing surfaces of twelve sections were laid on a broken stone foundation, five built by the penetration and seven by the mixing method. In all but one section built by the mixing method the stone was heated. The remaining four are made up as follows: Hassam Portland Cement Concrete Pavement, water-bound macadam, Sicilian Asphalt Company's Asphalto, which in appearance resembles ground Kentucky Rock Asphalt, and a surface application of asphalt binder and sand on an old macadam strip. In all but one section, where paving gravel was used in the wearing surface, the stone, both for foundation and wearing surface, was Clinton Point crushed trap rock.

The section built with the surface application and those built by the penetration methods are discussed under Topics Nos. 5 and 6, respectively.

These experiments were made with a view to determine by a service test a suitable medium-priced pavement for suburban residential streets, and also a cheaper grade of pavement for country roads and parkways, where the constantly increasing automobile traffic has made it imperative to lay something better than a water-bound macadam pavement. Therefore, it is to be hoped that the information obtained through observation of these experiments will be of value, both from an engineering and an economic point of view. The maintenance cost of water-bound macadam subjected to automobile traffic is appalling, and consequently it will be necessary in the future to construct a pavement where, even though the initial cost may be a little higher, the maintenance cost will be very much lower.

Accurate cost data of the construction of the various sections have been kept, and cost data of the maintenance will be also be kept. Analyses of the bituminous binders have been made by the United States Office of Public Roads, in accordance with their methods of

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testing bituminous binders, and in accordance with the recommendations of the Special Committee of the American Society of Civil Engineers on "Bituminous Materials for Road Construction," and levels have been taken which will make it possible to ascertain the amount of wear at any time. Mr. Connell.

It is the intention to take a traffic census of the road once each month up to December, 1911. The census will be taken on Fridays, Saturdays, Sundays, and Mondays, in order to take into account two holidays and two normal week days. Due to the fact that the speaker considers that these sections should undergo a year's use before any very definite knowledge as to their relative merits can be expressed, he does not desire to report on their present condition.

The White Plains Road is a wide thoroughfare extending northeastward through the northerly section of the Borough of the Bronx to Mount Vernon, and on toward White Plains. The experimental roadway is laid on the east side of the road, and extends from the intersection of White Plains Avenue and the Boston Road to Burke Street, a distance of 4 120 ft. The maximum grade is 1% and the crown is $\frac{3}{8}$ in. to 1 ft. No streets intersect White Plains Road between the Boston Road and Burke Street, therefore all the sections will be subjected to the same amount of traffic. A double-track electric railway runs along White Plains Avenue, and the pavement is carried up to the outside girder rail of the track.

The water-bound macadam section was laid as a basis of comparison, both with regard to wear and cost of construction. The cost of experimental work of this character, where the sections are short and all differ in methods of construction, is necessarily high, as the organization can hardly be perfected in making satisfactory progress before the section is completed. Therefore it may be safe to assume that the excess cost of the water-bound macadam may be used to determine the percentage to be deducted from the actual cost of any other section, in order to ascertain its normal cost. The total cost per square yard of the ordinary macadam, 6 in. deep, as laid on this road, was \$0.771, which is about 25% in excess of the average cost.

Work on the experimental road was begun on September 29th, and was completed on November 22d, 1910. The number of sections under construction at the same time ranged from two to nine. It will be observed, from the lateness of the season and daily record of the temperature taken during the period of construction, that the work was not done under the most favorable conditions. The temperature when the work was in progress varied from 40 to 79° Fahr., the average being 57° Fahr. However, it was thought better to make a start and finish this season than wait about 9 months for more perfect conditions.

Mr. Connell.

It is to be observed that the labor item is somewhat high. This is partly due to the rate of pay, namely, \$2.25 for an 8-hour day, and partly to the fact that the style of work was entirely new to the foreman as well as to the men. There are several points which stand out in connection with the work, which it may be worth while noting, namely: the comparatively low cost of Section 5, where the binder was mixed with cold stone in a concrete mixer; the high cost per square yard for fuel where the stone was heated; the additional cost per square yard for mixing tar and asphalt; the excess of binder used in the seal coat of several sections; the high cost of handling the Amiesite pavement, due entirely to the paving mixture being stiff, owing to cold weather; the difference in the cost of the concrete foundations laid by the mixing method and that by the process of spreading cement and screenings mixed, sprinkling and rolling, as in water-bound macadam; and the high cost of labor, namely, \$0.054 per sq. yd., for spreading and rolling the broken stone foundation course, due to re-handling the material.

The methods of construction used on White Plains Road, both for penetration work and the mixing method, were rather crude, and unless a suitable pressure machine is used for penetration work which will insure an equal distribution of bituminous binder, and unless an economical mixer is used for heating and mixing the stone and bituminous binder, thereby decreasing the labor cost, the cost of work of this character will be unnecessarily high in comparison with water-bound macadam. The method of construction, therefore, should be taken into consideration in comparing the relative cost of the various sections with the water-bound macadam.

A description of the methods of construction, the binders used, and cost data relative to the various sections constructed by the mixing method, follows. A traffic census taken in January, 1911, and a table of temperatures, are also appended.

White Plains Avenue Experimental Pavements.

Section One: 0 + 00 to 3 + 25.

Bituminous Pavement, Mixing Method, laid by the Barber Asphalt Company.

Foundation Course.—A concrete foundation was laid, to a finished depth of 4½ in., in the proportions of 1:3:6.

Surface Course or Paving Mixture.—The paving mixture consists of the following mineral aggregate by weight, 73% paving gravel, 19½% sand, 1½% dust, and 5.4% Bermudez Asphalt, 55° penetration. The mixing was done at the Barber Asphalt Plant, then laid at a temperature of 220° and rolled to a finished depth of 1¾ in. with a 10-ton roller.

Seal Coat.—From 0 + 00 to 1 + 52, ½ gal. of Asphaltic Cement per sq. yd. was applied, over which sand was spread. Mr. Connell.

COST PER SQUARE YARD.

Concrete foundation	$0.639
Surface course	0.715
Seal coat	0.085
	$1.439

DETAILED COST PER SQUARE YARD.

Concrete Base, 4½ in. thick.

		Per square yard.
Labor	$168.00	$0.219
1¼-in. stone, 86 cu. yd., at $1.65	141.90	0.185
Sand, 43 " " " 1.21	52.03	0.068
Cement, 95 bbl., " 1.35	128.25	0.167
767 sq. yd., at $0.639	$490.18	$0.639

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	24	$5.00	$15.00
Laborers	22	528	1.75	115.50
Watchmen	2	48	1.75	10.50
Timekeeper	1	24	3.00	9.00
Team	1	24	6.00	18.00
				$168.00

SURFACE COURSE.

The cost was about 71½ cents per sq. yd., estimated from the cost of Bronx 55° penetration, deducting $27 for the difference in the cost of gravel and trap rock.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	8	$5.00	$5.00
Laborers	12	96	1.75	21.00
Watchman	1	8	1.75	1.75
Tampers	4	32	2.50	10.00
Rakers	3	24	2.50	7.50
Smoothers	2	16	2.00	4.00
Rollerman	1	8	4.50	4.50
Timekeeper	1	8	3.00	3.00
767 sq. yd., at $0.074				$56.75

SEAL COAT.

		Per square yard.
Labor	$9.22	$0.026
Chips, 5 cu. yd., at $1.90	9.50	0.027
Binder, 89 gal., at $0.131	11.66	0.032 = 0.248 gal. per sq. yd.
359 sq. yd., at $0.085	$30.38	$0.085

Mr. Connell.

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	4	$5.00	$2.50
Pouring asphalt	2	6	2.25	1.68
Squeegee	1	3	2.25	0.84
Heating asphalt	1	3	2.25	0.84
Carrying asphalt	2	6	2.25	1.68
Spreading chips	2	6	2.25	1.68
				$9.22

The actual time of laying the surface course was 8 hours, or an average of 767 sq. yd. per day. The actual time of laying the seal coat was 3 hours, or an average of 957 sq. yd. per day.

SECTION TWO: 3 + 25 to 6 + 50.

Bituminous Pavement constructed by the Mixing Method and laid in accordance with Borough of The Bronx Specifications.

Foundation Course.—This consists of 1½-in. crushed trap rock, thoroughly rolled with an 18-ton roller to a finished depth of about 3¾ in.

Surface Course or Paving Mixture.—The paving mixture is composed of two parts of crushed trap rock passing through a 1¼-in. ring and not containing more than 5% of dust, one part of sand and 7.4% by weight of Bermudez Asphalt, 55° penetration. The mixing was done at an asphalt plant, and the paving mixture was laid at a temperature of 220° and rolled to a finished depth of 2½ in. with a 10-ton roller.

Seal Coat.—From 3 + 25 to 4 + 78, ¼ gal. of Asphaltic Cement per sq. yd. was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.283
Surface course	0.751
Seal coat	0.081
	$1.115

DETAILED COST PER SQUARE YARD.

		Per square yard.
106.4 cu. yd. trap rock, at $1.65	$175.56	$0.229
Labor	....	0.054
		$0.283

SURFACE COURSE.

					Per square yard.	
Mixing at plant				$110.48	$0.144	
Labor				51.25	0.067	
Hauling				93.84	0.122	
Asphalt cement	1 600 gal.	at	$0.131	209.60	0.273	= 2.09 gal. per sq. yd.
¾-in. stone	54 cu. yd.	"	1.25	67.50	0.088	
Sand	30½ " "	"	0.75	22.87	0.030	
Dust	2 760 lb.	"	3.50	4.83	0.006	
Grit	3 cu. yd.	"	1.25	3.75	0.005	
Coal	8 030 lb.	"	3.00	12.05	0.016	
767 sq. yd., at $0.751				$576.17	$0.751	

Mr Connell.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	8	$5.00	$5.00
Laborers	8	64	1.75	14.00
Watchmen	3	24	1.75	5.25
Tampers	4	32	2.50	10.00
Rakers	3	24	2.50	7.50
Smoothers	1	8	2.00	2.00
Rollerman	1	8	4.50	4.50
Timekeeper	1	8	3.00	3.00
767 sq. yd., at $0.067				$51.25

SEAL COAT.

			Per square yard.	
Labor		$9.52	$0.026	
Chips	3 cu. yd., at $1.90	5.70	0.016	
Asphalt cement	95 gal. " 0.13	12.35	0.035	= 0.26 gal. per sq. yd.
Wood	¼ cord " 6.50	1.52	0.004	
361 sq. yd., at $0.081		$29.19	$0.081	

LABOR.

	No.	Hours.	Rate.	Totals.
Heating asphalt	1	11	$1.75	$2.38
Pouring "	1	11	1.75	2.38
Spreading chips	2	22	1.75	4.76
				$9.52

The actual time of laying the surface course was 8 hours, or an average of 767 sq. yd. per day.

SECTION THREE: 6 + 50 to 8 + 00.

Bituminous Pavement, Mixing Method.

Foundation Course.—A concrete base was laid, under the supervision of Mr. C. I. Williams, of Utica, N. Y., to a finished depth of 4¾ in.; 1½-in. stone was spread to a depth of 6 in., then screenings and cement mixed dry in proportions of 3:1 were spread, rolled, and sprinkled with water. The operation was repeated until the voids were filled. An 18-ton roller was used.

Paving Mixture or Surface Course.—The paving mixture is composed of 2 parts of crushed trap rock passing through a 1¾-in. ring and not containing more than 5% of dust, 1 part of sand, and 8% of Bermudez asphalt, 55° penetration. The mixing was done at an asphalt plant, and the paving mixture was laid at a temperature of 220° and rolled to a finished depth of 2 in. with a 10-ton roller.

Seal Coat.—The surface course was painted with about ¼ gal. of Asphaltic Cement per sq. yd., over which chips were spread and rolled.

Mr. Connell.

Cost per Square Yard.

Concrete foundation	$0.502
Surface course	0.90
Seal coat	0.08
	$1.482

Detailed Cost per Square Yard.

Concrete Base, 4¾ to 5 in. thick.

			Per square yard.
Labor		$19.96	$0.056
1¼-in. stone 59 cu. yd., at $1.65		97.35	0.275
Screenings 9.0 " " " 1.65		14.85	0.042
Cement 20¼ bbl. " 1.35		27.34	0.077
354 sq yd. at	$0.45	$159.50	$0.45
Spreading 1¼-in. stone	0.052		
Total	$0.502		

Labor.

	No.	Hours.	Rate.	Totals.
Foremam	1	9	$4.00	$4.50
Laborers	5	43	2.25	12.09
Rollerman	1	6	4.50	3.37
				$19.96

Surface Course.

		Cost per square yard.	
Mixing at plant	$105.14	$0.297	
Labor in field	32.06	0.090	
Hauling	40.80	0.115	
Asphalt cement, 742.5 gal., at $0.13	96.52	0.273	= 2.1 gal. per sq. yd.
Trap rock 22.5 cu. yd., at 1.25	28.12	0.079	
Sand 9.0 " " 0.75	6.75	0.020	
Dust 1 440 lb., " 3.50	2.52	0.008	
Coal, soft, 5 300 lb. " 3.00	7.95	0.022	
" hard, 400 " " 6.00	1.20	0.003	
354 sq. yd., at $0.907	$321.06	$0.907	

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	6	$5.00	$3.75
Laborers	8	48	1.75	10.50
Watchman	1	6	1.75	1.31
Tampers	2	12	2.50	3.75
Rakers	3	18	2.50	5.62
Smoothers	1	6	2.00	1.50
Rollerman	1	6	4.50	3.38
Timekeeper	1	6	3.00	2.25
354 sq. yd., at $0.090				$32.06

Mr. Connell.

SEAL COAT.

			Per square yard.
Labor		$8.80	$0.025
Chips	3 cu. yd., at $1.90	5.70	0.016
Asphalt cement, 82.5 gal.,	" 0.13	10.72	0.030 = 0.23 gal. per sq. yd.
Wood, ½ cord	" 6.50	3.25	0.010
354 sq. yd., at $0.080		$28.47	$0.081

LABOR.

	No.	Hours.	Rate.	Totals.
Heating asphalt	1	6	$1.75	$1.31
Pouring asphalt	2	12	1.75	2.62
Spreading chips	2	12	1.75	2.62
Timekeeper	1	6	3.00	2.25
				$8.80

The actual time of laying the surface course was 5 hours, or an average of 454 sq. yd. per day.

SECTION FOUR: 8 + 00 to 11 + 25.

Bituminous Pavement, Mixing Method, laid under the supervision of the Standard Oil Company.

Foundation Course.—This consists of 1½-in. crushed trap rock thoroughly rolled with an 18-ton roller to a finished depth of about 3¾ in.

Surface Course or Paving Mixture.—The paving mixture consists of the following mineral aggregate by weight: 58% of ¾-in. crushed trap rock, 20% of ⅜-in. chips, 15% of sand, and 7% of Standard Oil Company's Special Asphalt Binder. The binder, stone, and sand were heated on the ground and mixed by hand on mixing boards; the mixture was spread and rolled to a finished depth of 2¾ in. with an 18-ton roller.

Seal Coat.—From 9 + 95 to 10 + 66, ⅓ gal. per sq. yd. was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.283
Surface course	0.964
Hauling binder	0.009
Fuel	0.063
Asphalt and stone heaters	0.004
	$1.323

DETAILED COST PER SQUARE YARD.

Base, 3¾ in. thick.

		Per square yard.
106.4 cu. yd. trap rock, at $1.65	$175.56	$0.229
Labor	....	0.054
		$0.283

Mr. Connell.

SURFACE COURSE.

		Per square yard.
Labor	$368.14	$0.480
¾-in. stone, 60 cu. yd., at $1.80	108.00	0.140
Chips, 18 cu. yd., at $1.90	34.20	0.045
Sand, 12 cu. yd., at $1.21	14.52	0.019
Binder, 2 688 gal., at $0.08	215.04	0.280 = 3.5 gal. per sq. yd.
767 sq. yd., at $0.964	$739.90	$0.964

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	24	$4.00	$12.00
	2	48	4.00	24.00
Spreading	1	24	2.25	6.75
	2	48	2.25	13.50
Mixing	8	192	2.25	54.00
	4	108	2.25	30.37
Wheeling	6	280	2.25	78.75
Heating and carrying oil	4	96	2.25	27.00
	5	120	2.25	33.75
Heating stone	5	235	2.25	66.09
Rollermen	3	39	4.50	21.93
767 sq. yd., at $0.480				$368.14

During the first 3 days one gang worked; during the last 3 days two gangs worked. The total quantity laid was 767 sq. yd., or an average of 85 sq. yd. per day per gang.

SECTION FIVE: 11 + 25 to 14 + 50.

Bituminous Pavement, Mixing Method, laid under the supervision of the Texas Company.

Foundation Course.—This consists of 1½-in. crushed trap rock, thoroughly rolled with an 18-ton roller to a finished depth of about 3¾ in.

Surface Course or Paving Mixture.—The paving mixture consists of ¾-in. trap rock and 6.7% by weight of Texas Macadam Binder. The binder was heated and mixed with the cold stone in a standard concrete mixer on the ground. The paving mixture was spread, and rolled with an 18-ton roller to a finished depth of 2¾ in.

Seal Coat.—From 12 + 36 to 13 + 03, a seal coat of Texas 55 Special (about 1¼ gal. per sq. yd.) was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.283
Surface course	0.721
Tar heater	0.001
Fuel	0.021
Hauling binder	0.006
	$1.032

Mr. Connell.

Detailed Cost per Square Yard.

Base, $3\frac{3}{4}$ in. thick.

		Per square yard.
108.6 cu. yd. trap rock, at $1.65	$179.19	$0.229
Labor	......	0.054
		$0.283

Surface Course.

		Per square yard.
Labor	$182.86	$0.234
¾-in. stone, 81 cu. yd., at $1.80	145.80	0.186
Binder, 1 728 gal., at $0.12	207.36	0.265 = 2.2 gal. per sq. yd.
Concrete mixer (rent), 7 days, at $4.00	28.00	0.036
782 sq. yd., at $0.721	$564.02	$0.721

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	44	$4.00	$22.00
Spreading	2	90	2.25	25.31
Feeding mixer	4	171	2.25	48.09
Wheeling	3	132	2.25	37.12
	3	25	2.25	7.03
Heating binder	1	48	2.25	13.50
Rollerman	1	9	4.50	5.06
Mixer engineer	1	44	4.50	24.75
782 sq. yd., at $0.234				$182.86

The actual time of laying the surface course was 5 days and 4 hours, or an average of 142 sq. yd. per day.

Section Six: 14 + 50 to 17 + 75.

Amiesite, laid under the supervision of the Amiesite Company.

Foundation Course.—This consists of $1\frac{1}{2}$-in. crushed trap rock and sufficient screenings to fill the voids. This was spread, and then rolled with an 18-ton roller to a finished depth of $3\frac{1}{2}$ in.

Surface Course or Paving Mixture.—The paving mixture was shipped from the Amiesite plant, and laid in two courses: the first course was composed of $1\frac{1}{2}$-in. stone with binder, etc.; laid and rolled to a finished depth of 2 in. The second or wearing course was composed of $\frac{5}{8}$-in. stone, binder, etc., laid and rolled to a finished depth of 1 in. The paving mixture, therefore, had a finished depth of 3 in. An 18-ton roller was used.

Cost per Square Yard.

Base course	$0.329
Surface Course—	
Labor, spreading	0.155
" unloading	0.132
Hauling	0.179
	$0.795*

* Does not include cost of Amiesite.

Mr. Connell.

Detailed Cost per Square Yard.

Base, $3\frac{1}{2}$ in. thick.

		Per square yard.
106.4 cu. yd. trap rock, at $1.65	$175.56	$0.229
21.3 " " screenings, at $1.65	35.11	0.046
Labor	....	0.054
		$0.329

Surface Course.

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	21	$4.00	$10.50
Laborers	17	356	2.25	100.12
Rollerman	1	15	4.50	8.43
766 sq. yd., at $0.155				$119.05
Screenings, 1½ cu. yd., at $1.65		$2.47	$0.003 per sq. yd.	

Labor Unloading 72 Cu. Yd. Amiesite from Railroad Cars.

	No.	Hours.	Rate.	Totals.
Foreman	1	34	$4.00	$17.00
Laborers	9	297	2.25	83.90
766 sq. yd., at $0.132				$100.90

Hauling from Railroad Depot.

	No.	Hours.	Rate.	Totals.
Teams	3	15	$5.00	$9.38
	7	56	5.00	35.00
	9	27	5.00	16.88
	6	21	5.00	13.12
	8	52	5.00	32.50
	3	12	5.00	7.50
	3	24	5.00	15.00
	4	13	5.00	8.12
766 sq. yd., at $0.179				$137.50

The actual time of laying the surface course was 2 days and 4 hours, or an average of 306 sq. yd. per day. The material was very hard, due to the length of time it remained in the cars, making the work of unloading from the cars and laying the surface very difficult. Under favorable conditions the labor should be reduced by 70 per cent.

SECTION SEVEN: 17 + 75 to 20 + 67. Mr. Connell.

Sicilian Asphalt, laid by the Sicilian Asphalt Company.

Foundation Course.—This consists of 1½-in. crushed trap rock, rolled with an 18-ton roller to a finished depth of 4½ in.

Surface Course or Paving Mixture.—The paving mixture is called "Asphalto," and was mixed at the Sicilian Asphalt Company's plant, hauled to White Plains Avenue and spread. The paving mixture was not rolled until comparatively cool, and then a Sicilian Asphalt 10-ton grooved roller was used. The finished depth was 1½ in. A light layer of cement was then sprinkled over the surface.

COST PER SQUARE YARD.

Base course	$0.382
Surface course—	
Field labor	0.030
Cement	0.001
Hauling	0.087
	$0.500*

DETAILED COST PER SQUARE YARD.

Base, 4½ in. thick.

		Per square yard.
129.9 cu. yd. trap rock, at $1.65	$214.33	$0.299
13.0 " " screenings, at $1.65	21.40	0.029
Labor	....	0.054
		$0.382

SURFACE COURSE.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	5	$5.00	$3.12
Spreading	10	50	1.75	10.93
Rakers	2	10	2.50	3.12
Spreading cement	1	6	1.75	1.31
Rollerman	1	6	4.50	3.37
718 sq. yd., at $0.030				$21.85
Cement, 2½ bags, at $1.35 per bbl		$0.840	$0.001 per sq. yd.	

HAULING.

19 loads, at $3.25 .... $61.75 $0.087 per sq. yd.

The actual time of laying the surface course was 5 hours, or an average of 1 148 sq. yd. per day.

SECTION EIGHT: 20 + 67 to 22 + 75.

Bituminous Pavement, Mixing Method, laid under the supervision of the Barrett Manufacturing Company.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, rolled with an 18-ton roller to a finished depth of 3¾ in.

* Cost of Asphalto not included.

Mr. Connell.

Surface Course or Paving Mixture.—The paving mixture consists of 3 parts of ¾-in. crushed trap rock, 1 part of chips and 6½% by weight of Tarvia X. The stone and Tarvia X were heated on the ground and mixed by hand on mixing boards, spread, and rolled with an 18-ton roller to a finished depth of 2¾ in.

Seal Coat.—About 0.6 gal. per sq. yd. of Tarvia X was applied, over which chips were spread and rolled.

Cost per Square Yard.

Base course	$0.305
Surface course	0.913
Seal coat	0.137
Hauling binder	0.009
Tar and stone heaters	0.002
Fuel	0.063
	$1.429

Detailed Cost per Square Yard.

Base, 3¾ in. thick.

		Per square yard.
68.1 cu. yd. trap rock, at $1.65	$112.36	$0.229
6.8 " " screenings, at $1.65	11.23	0.022
Labor	....	0.054
		$0.305

Surface Course.

		Per square yard.	
Labor	$206.60	$0.421	
1½-in. stone, 6 cu. yd., at $1.65	9.90	0.177	
¾-in. " 43 " " 1.80	77.40		
Chips, 14 " " 1.90	26.60	0.054	
Binder, 1 344 gal., at 0.095	127.68	0.261	= 2.7 gal. per sq. yd.
491 sq. yd., at $0.913	$448.18	$0.913	

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	42	$4.00	$21.00
Spreading	1	42	2.25	11.81
Mixing	4	169	2.25	47.53
Wheeling	3	142	2.25	39.93
Heating binder	2	94	2.25	26.43
Heating stone	3	150	2.25	42.19
	4	29	2.25	8.15
Rollerman	1	17	4.50	9.56
491 sq. yd., at $0.421				$206.60

Seal Coat.

		Per square yard.	
Labor	$33.24	$0.068	
Chips, 3.5 cu. yd., at $1.90	6.65	0.014	
Binder, 288 gal., at $0.095	27.36	0.055	= 0.58 gal. per sq. yd.
491 sq. yd., at $0.137	$67.25	$0.137	

Mr. Connell.

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	8	$4.00	$4.00
Pouring binder	1	8	2.25	2.25
Squeegee	1	8	2.25	2.25
Heating and carrying binder	3	26	2.25	7.31
Spreading chips	2	16	2.25	4.50
Wheeling and heating chips	4	32	2.25	9.00
Rollerman	1	7	4.50	3.93
				$33.24

The actual time of laying the surface course was 6 days and 2 hours, or an average of 78½ sq. yd. per day. The actual time of laying the seal coat was 1 day, or an average of 491 sq. yd. per day. The chips required heating, as they were wet and cold.

SECTION NINE: 22 + 75 to 24 + 50.

Bituminous Pavements, Mixing Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3¾ in.

Surface Course or Paving Mixture.—From 22 + 75 to 23 + 02, the binder consists of a mixture of tar and 10% of refined Bermudez asphalt. From 23 + 02 to 24 + 00, the binder consists of a mixture of tar and 15% of Bermudez asphalt of 60° penetration. From 24 + 00 to 24 + 40, the binder consists of a mixture of tar and 15% of Bermudez apshalt of 110° penetration. From 24 + 40 to 24 + 50, Bermudez asphalt of 110° penetration was used for a binder. The paving mixture consists of 3 parts of ¾-in. stone, 1 part of chips, and 7% by weight of bitumen. The stone and binder were heated on the ground, mixed by hand on mixing boards, spread, and rolled with an 18-ton roller to a finished depth of 2¾ in.

Seal Coat.—About 0.6 gal. of asphalt per sq. yd. was applied, over which chips were spread and rolled.

COST PER SQUARE YARD.

Base course	$0.305
Surface course	1.034
Seal coat	0.146
Fuel	0.063
Hauling binder	0.009
Tar and stone heaters	0.002
Mixing binder	0.076
	$1.635

DETAILED COST PER SQUARE YARD.

Base, 3¾ in. thick.

		Per square yard.
57.3 cu. yd. trap rock, at $1.65	$94.54	$0.229
5.7 " " screenings, " 1.65	9.45	0.022
Labor	....	0.054
		$0.305

Mr. Connell.

Surface Course.

			Per square yard.
Labor		$197.84	$0.479
1½-in. stone,	5 cu. yd., at $1.65	8.25	0.181
¾-in. "	37 " " " 1.80	66.60	
Chips,	12 " " " 1.90	22.80	0.055
Binder,	1 115 gal., " 0.118	131.57	0.319 = 2.7 gal. per sq. yd.
413 sq. yd., at $1.034		$427.06	$1.034

Field Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	47	$4.00	$23.50
Spreading	1	46	2.25	12.93
Mixing	4	184	2.25	51.75
Wheeling	3	146	2.25	41.06
Heating binder	4	192	2.25	53.98
	3	18	2.25	5.06
Rollerman	1	17	4.50	9.56
413 sq. yd., at $0.479				$197.84

Seal Coat.

		Per square yard.
Labor	$23.50	$0.056
Chips, 3 cu. yd., at $1.90	5.70	0.014
Binder, 264 gal., at $0.118	31.15	0.076 = 0.64 gal. per sq.yd.
413 sq. yd., at $0.146	$60.35	$0.146

Labor, Mixing Tar and Asphalt on Ground.

Foreman	3 hours	$1.50
Laborers	2 men, 9 hours	2.53
"	3 " 24 "	6.75
288 gal., at $0.037		$10.78

This labor included part of two days, requiring heating up twice and emptying from kettles to barrels for future use. The material should be mixed as used, requiring only an extra kettle kept hot, with asphalt in it ready to mix.

Labor.

	No.	Hours.	Rate.	Totals.
Foreman	1	6	$4.00	$3.00
Pouring	1	6	2.25	1.68
Squeegee	1	6	2.25	1.68
Heating and carrying binders	3	19	2.25	5.34
Spreading chips	2	12	2.25	3.37
Wheeling and heating chips	4	20	2.25	5.62
Rollerman	1	5	4.50	2.81
Total				$23.50
Mixing binder at plant, $31.56 = $0.076 per sq. yd.				

The cost of this section is high because the work was not continuous, due to waiting for binder and also due to cold weather, which made the stone harder to heat. The chips were wet and cold, and required heating. Mr. Connell.

The actual time of laying the surface course was 6 days, or an average of 69 sq. yd. per day. The actual time of laying the seal coat was 6 hours, or an average of 551 sq. yd. per day.

Section Ten: 24 + 50 to 27 + 97.

Bituminous Pavement, Mixing Method.

Foundation Course.—This consists of 1½-in. crushed trap rock and screenings, spread and rolled to a finished depth of 3¾ in.

Surface Course or Paving Mixture.—The paving mixture consists of 3 parts of ¾-in. trap rock, 1 part of ⅜-in. trap rock chips, and 6% by weight of refined tar. The binder and stone were heated on the ground, and mixed by hand on mixing boards, spread, and rolled with an 18-ton roller to a finished depth of 2¾ in. The binder used was refined tar made up in accordance with specifications drawn up by the Office of Public Roads, of the United States Department of Agriculture.

Seal Coat.—About 0.6 gal. per sq. yd. was applied, over which chips were spread and rolled.

Cost per Square Yard.

Base course	$0.305
Surface course	0.868
Seal coat	0.135
Fuel	0.063
Hauling binder	0.008
Tar and stone heaters	0.003
	$1.382

Detailed Cost per Square Yard.

Base, 3¾ in. thick.

		Per square yard.
113.6 cu. yd. trap rock, at $1.65	$187.44	$0.229
11.3 " " screenings, " 1.65	18.74	0.022
Labor		0.054
		$0.305

Surface Course.

		Per square yard.
Labor	$308.91	$0.377
¾-in. stone, 82.5 cu. yd., at $1.80	148.50	0.181
Chips, 27.5 cu. yd., at $1.90	52.25	0.064
Binder, 2 016 gal., at $0.10	201.60	0.246 = 2.46 gal. per sq. yd.
819 sq. yd., at $0.868	$711.26	$0.868

Mr. Connell.

FIELD LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	52	$4.00	$26.00
Spreading	1	20	2.25	5.62
	2	63	2.25	17.71
Mixing	4	201	2.25	56.53
Wheeling stone	4	99	2.25	27.84
	5	140	2.25	39.37
Heating stone	3	36	2.25	10.12
	5	200	2.25	56.25
Heating tar	5	176	2.25	49.50
	3	23	2.25	6.47
Rollerman	1	24	4.50	13.50
819 sq. yd., at $0.868				$308.91

SEAL COAT.

		Per square yard.
Labor	$50.89	$0.062
Chips, 6.0 cu. yd., at $1.90	11.40	0.014
Binder, 480 gal., " 0.10	48.00	0.059 = 0.59 gal. per sq. yd.
819 sq. yd., at $0.135	$110.29	$0.135

LABOR.

	No.	Hours.	Rate.	Totals.
Foreman	1	9	$4.00	$4.50
Boring	1	10	2.25	2.81
Squeegee	1	10	2.25	2.81
Heating tar	6	54	2.25	15.18
Carrying tar	4	35	2.25	9.84
Spreading chips	1	9	2.25	2.53
Wheeling chips	4	31	2.25	8.72
Rollerman	1	8	4.50	4.50
819 sq. yd., at $0.062				$50.89

The actual time of laying the surface course was 6 days and 4 hours, or an average of 126 sq. yd. per day. The actual time of laying the seal coat was 1 day and 2 hours, or an average of 655 sq. yd. per day. The chips were not heated.

COST OF BASE COURSE, FUEL, HAULING BINDER, TAR KETTLES, STONE HEATERS, ETC.

3¾-in. Macadam Base.		Per square yard.
379.5 cu. yd. 1½-in. trap rock, at $1.65	$626.17	$0.229
37.9 " " screenings, " 1.65	62.61	0.022
Labor		0.054
2 733 sq. yd		$0.305

Mr. Connell.

3¾-in. Macadam Base.		Per square yard.
221.3 cu. yd. 1½-in. trap rock, at $1.65	$365.14	$0.229
44.2 " " screenings, " 1.65	73.02	0.046
Labor		0.054
1 592 sq. yd.		$0.329

3¾-in. Macadam Base.		Per square yard.
321.4 cu. yd. 1½-in. trap rock, at $4.165	$530.31	$0.229
Labor		0.054
2 316 sq. yd.		$0.283

4½-in. Macadam Base.		Per square yard.
129.9 cu. yd. 1½-in. trap rock, at $1.65	$214.33	$0.299
13.0 " " screenings, " 1.65	21.43	0.029
Labor		0.054
718 sq. yd.		$0.382

4¾-in. Macadam Base.		Per square yard.
59 cu. yd. 1½-in. trap rock, at $1.65	$97.35	$0.275
Labor		0.054
354 sq. yd.		$0.329

Spreading Base Course.

Foreman, 98 hours, at 50 cents	$49.00
Laborers, 1 149 " " 28⅛ "	323.15
Rollerman, 82 " " 56¼ "	46.12
7 713 sq. yd., at $0.054	$418.27

Fuel.

41 cords, at $6.50 .... $266.50
$0.048 per sq. yd. on 5 568 sq. yd.
Estimated: $0.021 per sq. yd. for penetration and mixing with cold stone.
" $0.063 " " " " mixing with hot stone.

The heating surface required to heat the stone was twice the area of that required to heat the tar and asphalt, therefore the mixing jobs when the stone was heated, required three times the quantity of wood used on penetration or cold stone sections, when the binder only was heated.

Hauling Binder.—The cost of hauling 12 000 gal. was $93.70 = $0.0078 per gal., the cost for the same haul should be: 5 loads = 500 gal. × 5 = 2 500 gal. per day, at $6.50 = $0.0026 per gal.

Tar Kettles and Stone Heaters.—Tar kettles cost about $100; their life is 10 years. Interest at 4% for 10 years, plus sinking fund = $8.006 per year for 10 years = $120.06. The total cost is $120.06 for 10 years = $12.006 per year; 120 working days = 10 cents per day.

Stone heaters of flat plates cost about $15.00; their life is 10 years; interest at 4% for ten years, plus sinking fund = $1.20 per year for 10 years = $18.00. Total cost $1.80 per year; 120 working days = 1½ cents per day.

Mr. Connell.

TABLE OF TEMPERATURES.

Pavement.	Date.	Temperature. 9 A. M.	12 NOON.	3 P. M.	Weather.
Barber Asphalt	Sept. 29, '10. { Begun ... Finished . }	62	65	67	Clear.
Bronx Asphalt, 55	Sept. 30, '10. Begun	62	65	67	Hazy.
	Oct. 1, '10. Finished	66	70	..	Clear.
Bronx Asphalt	Oct. 14, '10. Begun	60	68	69	Fair.
	Oct. 15, '10. Finished	61	63	..	Cloudy.
S. O. Mixture	Nov. 9, '10. Begun	39	45	46	Cloudy.
	Nov. 10, '10	48	54	55	Clear.
	Nov. 11, '10	44	48	43	Cloudy.
	Nov. 12, '10	35	37	..	Cloudy.
	Nov. 14, '10	38	44	48	Cloudy.
	Nov. 15, '10. Finished	33	39	40	Cloudy.
Texas Oil Mix	Oct. 5, '10. Begun	71	74	76	Cloudy.
	Oct. 6, '10	72	76	79	Clear.
	Oct. 8, '10	54	55	..	Cloudy.
	Oct. 10, '10	56	59	52	Clear.
	Oct. 11, '10	59	66	69	Clear.
	Oct. 13, '10	48	52	55	Clear.
	Oct. 14, '10	60	68	69	Fair.
	Oct. 15, '10. Finished	61	63	..	Cloudy.
Amiesite	Nov. 7, '10 Begun	34	40	43	Clear.
	Nov. 9, '10	39	45	46	Cloudy.
	Nov. 10, '10	48	54	55	Clear.
	Nov. 11, '10. Finished	44	48	43	Cloudy.
Sicilian Asphalt	Oct. 19, '10. Begun	59	65	68	Clear.
	Oct. 21, '10. Finished	55	57	59	Cloudy.
Tarvia Mixture	Oct. 26, '10. Begun	49	54	56	Clear.
	Oct. 27, '10	58	65	67	Cloudy.
	Oct. 28, '10	54	55	56	Clear.
	Oct. 29, '10	44	48	..	Clear.
	Oct. 31, '10	40	49	53	Clear.
	Nov. 1, '10. Finished	44	53	56	Clear.
(Flush Coat)	Nov. 10, '10	48	54	55	Clear.
	Nov. 11, '10	44	48	43	Cloudy.
Tar and Asphalt Mixture	Oct. 26, '10. Begun	49	54	56	Clear.
	Oct. 27, '10	58	65	67	Cloudy.
	Oct. 31, '10	40	49	53	Clear.
	Nov. 1, '10	44	53	56	Clear.
	Nov. 2, '10	52	57	60	Cloudy.
	Nov. 7, '10	34	40	43	Clear.
	Nov. 9, '10	39	45	46	Overcast.
	Nov. 10, '10	48	54	55	Clear.
U. S. Tar	Oct. 21, '10. Begun	55	57	59	Cloudy.
	Oct. 22, '10	52	53	..	Rain.
	Oct. 24, '10	52	54	56	Clear.
	Oct. 25, '10	55	60	60	Cloudy.
	Oct. 26, '10	49	54	56	Clear.
	Oct. 27, '10	58	65	67	Cloudy.
	Oct. 28, '10	54	55	56	Clear.
	Oct. 29, '10	44	48	..	Clear.
	Oct. 31, '10. Finished	40	49	53	Clear.

TRAFFIC CENSUS, WHITE PLAINS ROAD. TAKEN BETWEEN THE HOURS OF 5 A. M. AND 9 P. M.

Mr. Connell.

	Friday, January 6th, 1911.	Saturday, January 7th, 1911.	Sunday, January 8th, 1911.	Monday, January 9th, 1911.
Horse without vehicle	..	1	4	2
Horse vehicle, light	15	36	37	34
Horse vehicle, heavy	48	51	5	48
2 horse vehicle, light	5	8	3	6
2 horse vehicle, heavy	38	59	..	45
3-horse vehicle	1	..	..	..
4-horse vehicle	4	2	..	..
2-passenger car	2	5	11	3
4 or 5-passenger car	12	27	51	21
6- or 7-passenger car	9	12	34	9
Freight truck, omnibus, etc	17	21	..	18
Traction engine	..	..	..	..
2-traction engine	..	..	..	..
Miscellaneous heavy traffic	..	..	..	..

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

EXPERIMENTS ON RETAINING WALLS AND PRESSURES ON TUNNELS.

Discussion.*

By J. R. Worcester, M. Am. Soc. C. E.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E. (by letter).—In reading Professor Cain's admirable paper relating to experiments on retaining walls, the writer has looked in vain for a word of caution as to the effect which time plays in modifying the condition of equilibrium within a mass of earth. The author evidently considers it necessary to allow (by using a factor of safety) for a possible lessening of the angle of friction on account of a change in the amount of moisture, and possible vibrations, but states that in a great majority of cases the greatest thrust will occur where the earth has been recently deposited. It would appear that he neglects a possibility, if not a probability, of a readjustment of the earth particles through the influence of time, by which the angle of friction is lessened if not wholly cancelled.

The theory that cohesion in the earth and frictional resistance on the back of the retaining wall account for the experimental results seems indisputable, but such experiments must needs be carried through in a reasonable time, and, in that respect, at least, must needs differ from actual constructions which are intended to be permanent.

It is well known that unbraced excavations can often be carried vertically to considerable depths in safety, but that not infrequently—as many have learned to their sorrow—such unbraced banks have subsequently caved in. The slides in the Culebra cut may be mentioned

*This discussion (of the paper by William Cain, M. Am. Soc. C. E., published in *Proceedings* for January, 1911, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion

Mr. Worcester.

as a similar illustration. To be sure, the delayed motion of the earth (or rock) may be attributed to the effect of moisture, but that does not invalidate the argument, as one always has to reckon with water.

It is also a matter of common knowledge that, in braced excavations, the pressure on the sheeting and bracing frequently increases in time to an extent enormously exceeding the original pressure. In many instances this has caused a failure long delayed.

Another instance of the effect of time is found in many retaining walls, in which a very slow motion has occurred, though the walls appeared to be entirely stable when first built.

A frictional or tangential force along the back of the wall may tend to prevent motion, but it is difficult to conceive of this tangential force being perpetually present on the back of a stationary wall after the back-filling has become settled and consolidated. In the interior of the mass of earth at rest the author admits that the reaction between particles is not along inclined lines, but rather that the lines of pressure are vertical and horizontal. The vertical lines of force are, of course, caused by gravity, and the horizontal lines by the tendency of particles to wedge in between those below and to spread them.

If this conception of the forces within a mass of earth is reasonable, it would seem as if it might also be extended to the pressure against an immovable vertical wall. One must then consider whether it is possible for a horizontal pressure to cause the wall to move, without changing the conditions and introducing the inclined stresses. It would seem within the bounds of possibility that a very minute motion might be produced, and that this would be followed by a readjustment of stresses in the earth by which the forces would gradually resume the horizontal direction.

The nature of the soil undoubtedly has much to do with this question. In some kinds of clay there appears to be a sort of viscosity, such as is frequently seen in pitch and other materials, or a tendency toward a slow flowing. No amount of pressure would cause a sudden motion, but time will effect a motion under a slight pressure or even the force of gravity alone. It appears that this condition is produced by the very minute particles, each moving individually into a position in which the surrounding forces balance. If one cuts a vertical face in such material one cannot force the exposed particles out of their position by crowding them from behind, but each in its turn will feel the pressure unbalanced and will slowly move out. This may not be true to the same extent, with granular materials of large diameter, but a familiar instance is seen in fine wet sand. If a small excavation is dug in a wet beach sand, the banks will stand vertically at first, but, by watching closely, one may see the particles, beginning at the foot of the bank where there is most water, gradually moving out,

Mr. Worcester. overcoming the force of cohesion, and ever tending to seek a condition with a level surface. It seems quite likely that a similar tendency would exist in almost all soils, to greater or less degree, though perhaps it might be safely neglected in a mass of hard, irregularly-shaped fragments of stone which could interlock.

The point which the writer wishes to make is that a word of caution should accompany this argument for the frictional and cohesive forces; that they cannot always be relied on; and that sometimes the Rankine theory may be better than the wedge theory in designing, even though it does not seem to fit the experimental results.

Another warning may not be amiss, in considering the safe thickness to allow for retaining walls, and that is the effect of frost, where the surface of the ground is level and likely to retain moisture. The swelling force of freezing, under these circumstances, may be more than sufficient to overcome the beneficial effects of both cohesion and friction. Presumably this must be provided for in the "factor of safety," and is in itself a justification for a very appreciable factor.

It may be well to emphasize the fact that a large part of the author's assumed factor of safety seems to be absorbed in keeping the resultant within the middle third of the base. The proportions between width of wall and height, determined on pages 77 and 78,* are such as to keep this resultant just within the base. If, with these same proportions of wall, the factor were assumed so that the resultant were within the middle third, it would be found to be nearer 1¼ than 3. The author's statement on page 79,* that he "does not advocate the middle-third limit method in design," is not wholly clear, but the implication is that the resultant should be well within this limit. In this case, it seems as if the factor of safety would be wholly absorbed in thus locating the resultant, and would leave nothing for other elements of uncertainty.

* *Proceedings*, Am. Soc. C. E., for January, 1911.

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PAPERS AND DISCUSSIONS

WATER PURIFICATION PLANT, WASHINGTON, D. C. RESULTS OF OPERATION.

Discussion.*

By Messrs. Allen Hazen, George A. Johnson, Morris Knowles, and George C. Whipple.

Allen Hazen, M. Am. Soc. C. E. (by letter).—This paper contains a most interesting and instructive record of the actual operation of a large filter plant, and also a record of a number of experiments. The author has described some useful arrangements for improving the efficiency or reducing the cost. Mr. Hazen.

The utility of raking, as an intermediate treatment between scrapings, seems to have been clearly demonstrated. Its practical effect is to allow a greater quantity of water to be passed between scrapings, thereby saturating—if the term may be used—the surface layer with clay and other fine matter before removing it, instead of taking it off when only a thin surface layer of it has been thus saturated.

The large proportion of the total purification that takes place in passing through three reservoirs successively, holding in the aggregate a quantity of water equal to about 7 days' use, is very striking. Taking all the records, the percentage remaining after passing through these reservoirs, is as follows:

Sediment for the year, 1909-1910, Table 2	17%
Turbidities, 5-year average, Table 3	25%
Bacteria, 5-year average, Table 4	24%
Bacteria, selected winter months with high numbers in the raw water	20%
Bacteria, selected summer months with high numbers in the raw water	2.5%

*This discussion (of the paper by E. D. Hardy, M. Am. Soc. C. E., published in *Proceedings* for December, 1910, and presented at the meeting of February 15th, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Hazen.

There is considerable seasonal fluctuation in the results of settling and filtering, as is shown in Table 21.

TABLE 21.—AVERAGE REMOVAL OF TURBIDITY AND BACTERIA BY WASHINGTON FILTERS FOR WHOLE PERIOD, ARRANGED BY SEASONS.

		Winter.	Spring.	Summer.	Fall.	Year.
Turbidity, in parts per million:	raw	136	96	144	42	105
	settled	33	28	27	15	26
	filtered	4	3	1	0.5	2
Percentage left from:	settling	24	29	19	36	25
	filtering	12	10	4	3	8
	both	3	1	0.3	1	2
Bacteria per cubic centimeter:	raw	16 600	4 150	4 100	1 960	6 700
	settled	6 300	980	160	270	1 940
	filtered	149	29	18	22	54
Percentage left from:	settling	38	24	4	14	29
	filtering	2.4	3.0	11.2	8.2	2.8
	both	0.90	0.70	0.44	1.12	0.81

The fluctuation in the efficiency of the plant as a whole by seasons is greater with the turbidity than with the bacteria. During the winter the effluent contains 3% of the turbidity of the raw water, and in summer only 0.3 per cent. Most of this difference is represented by the increased efficiency of the filters in summer, and only a little of it by the increased efficiency of settling. With bacteria, on the other hand, the seasonal fluctuation of the plant as a whole is comparatively small, but the settling and storage processes are much more efficient in summer than in winter, the filters being apparently less efficient. The writer believes that they are only apparently less efficient, and not really so, the explanation being that some bacteria always grow in the under-drains and lower parts of the filter, and are washed away by the effluent. The average number of bacteria in summer in the settled water is 160 per cu. cm. and in the filtered water 18. These are very low numbers. It is the writer's view that nearly all of these 18 represent under-drain bacteria, and practically bear no relation to those in the applied water, and, if this view is correct, the number of bacteria actually passing through the various processes is at all times less than the figures indicate. In the warmer part of the year the difference is a wide one, and the hygienic efficiency of the process is much greater than is indicated by the gross numbers of bacteria.

The reduction of the typhoid death rate has not been as great with the change in water supply as was the case at Lawrence, Albany, and other cities, apparently because the Potomac water before it was filtered was not the cause of a large part of the typhoid fever.

The sewage pollution of the Potomac is much less than that of the Merrimac and the Hudson, and it is perhaps not surprising that this

relatively small amount of pollution was less potent in causing typhoid fever than the greater pollution of rivers draining more densely populated areas. Mr. Hazen.

The method of replacing the washed sand hydraulically seems to have worked better than could have been reasonably anticipated, and the writer believes that this was due, in part, to the excellent method of manipulation described in the paper. It is his feeling, however, that part of the success is attributable to the very low uniformity coefficient of the sand. In other words, the sand grains are nearly all of the same size, due to the character of the stock from which the filter sand was prepared; and, therefore, there is much less opportunity for separation of the sand according to grain sizes than there would be with the filter sand which has been available in most other cases.

Filter sand with a uniformity coefficient as low as that obtained at Washington has been rarely available for the construction of sand filters, and while the method of hydraulic return should certainly be considered, it will not be safe to assume that equally favorable results may be obtained with it with sands of high uniformity coefficients until actual favorable experience is obtained.

The writer believes that in calculating the cost of the water used in the plant itself the price chosen by the author, covering only the actual operating expenses of pumping and filtering, is too low. The capacity of the whole Washington Aqueduct system is reduced by whatever quantity is used in this way, and, in calculating the cost of sand handling, the value of the water used should be calculated on a basis which will cover the whole cost of the water, including all capital charges, depreciation, operating expenses, and all costs of every description. On this basis the water used in the sand-handling operations would probably be worth five or more times the sum mentioned by the author.

The cost of operation of the plant has come within the estimates made in advance, and has certainly been most reasonable. The cost of filter operations has averaged only about 50 cents per million gallons, and is so low that it is obvious that the savings which may be made by introducing further labor-saving appliances would be relatively small. It will be remembered that ten or fifteen years ago the cost of operating such filters under American conditions was commonly from \$2 to \$5 per million gallons.

The experiments represented by Tables 17 to 19, inclusive, serve to show that preliminary filtration, or multiple filtration, or any system of mechanical separation is incapable of entirely removing the finer clay particles which cause the residual turbidity in the effluent. They also show that this turbidity may be easily and certainly removed by the application of coagulant to the raw water during the occasional periods when its character is such as to require it.

Mr. Hazen.

These general propositions were understood by those responsible for the original design of the plant, as is shown by the author's quotations. These experiments, however, were necessary in order to demonstrate and bring home the conditions to those who thought differently, and who believed that full purification could be obtained by filtration alone, or by double filtration, without recourse to the occasional use of coagulant.

The experiments briefly summarized in Table 20 are of the greatest interest and importance. Six small filters, otherwise alike and like the large filters, all received the same raw water and were operated at different rates to determine the effect of rate on efficiency.

That the experimental results from the filter operating at the same rate as the large filters were on the whole somewhat inferior to those from the large filters for approximately the same period, may be attributed to the fact that the experimental filter was new while the large filters had been in service for some time and had thereby gained in efficiency. The greatest difference was in the *coli* results in Table 20, where it is shown that 24% of the 10-cu. cm. effluent samples from the experimental filter contained *coli*, in comparison with only from 1 to 3% of such samples from the main filters.

The results from the experimental filter operating at a rate of 1 000 000 gal. per acre daily may fairly be excluded, as the effluent probably contained more under-drain bacteria in proportion than filters operated at higher rates. The number of bacteria in the filter operating at a 3 000 000-gal. rate were 1.7% of those in the applied water; for the filter operating twice as fast, the percentage was 2.4; and, for the one operating more than ten times as fast, was only 3.0; thus indicating a surprisingly small increase in the number of bacteria with increase in rate.

Further and more detailed study by the writer of the unpublished individual results, briefly summarized in Table 20, confirms the substantial accuracy of the comparison based on the average figures as stated in that table.

It must be kept in mind, in considering these results, that the number of bacteria in each case is made up of two parts, namely, those coming through the filter—which number is presumably greater as the rate is greater—and, second, those coming from harmless growths in the under-drains and lower parts of the filter—the numbers of which per cubic centimeter are presumably less as the rate is greater—and these two parts, varying in opposite directions, may balance each other, as they seem to do in this case, through a considerable range. It may thus be that the number of bacteria really passing the filter varies much more with the rate than is indicated by the gross results.

It is also of interest to note that the sand filter (called a preliminary filter) in Table 18, filled with the same kind of sand, when

operated at an average rate of 50 000 000 gal. per acre daily for a year, allowed 18% of the applied bacteria to pass, in comparison with 3% found in Filter No. 6 of Table 20, operated at an average rate of 38 000 000 gal. per acre daily. Mr. Hazen.

There was one point of difference in the manipulation: the preliminary filter was washed by a reversed current of water, as mechanical filters are washed, while Filter No. 6 was cleaned by scraping off the surface layer, as is usual with sand filters. Whether the great difference in bacterial results with a relatively small difference in rate is to be attributed to this difference in manipulation the writer will not undertake to state.

If the experimental results of Table 20 indicate correctly the conditions which obtain in filtering Potomac water, then increasing the rate of filtration so as to double it, or more than double it, would make but little difference in the quality of the effluent as measured by the usual bacterial methods. If the increase in rate were accompanied by the preliminary filtration of the water, then, presumably, there would be little change in the quality of the effluent, and the maintenance of excellent results might be incorrectly attributed to the influence of the preliminary filter.

It would also seem that the apparatus which is sometimes used for determining and controlling the rate with more than the ordinary degree of precision is hardly justified by such experimental results as those presented by the author.

In contrast to these results may be mentioned those obtained by Mr. H. W. Clark,* for experimental filters operated with Merrimac River water, at rates ranging from 3 000 000 to 16 000 000 gal. per acre daily. The results are the average of nearly two years of experimental work, the period having been nearly coincident with that covered by the author's experiments, and of many hundreds of bacterial analyses of each effluent, and form, with the author's experiments, the most thorough-going studies of the effect of rate on efficiency that have come to the writer's attention.

Mr. Clark's results are given in Table 22.

TABLE 22.

Effective size of sand.	Filter No.	Rate, in gallons per acre daily.	Bacteria per cubic centimeter in effluent.	Bacterial efficiency.	*B. Coli* in 1 cu. cm. (percentage of positive tests).
0.28	A	3 000 000	48	99.1	5.0
0.25	B	5 000 000	85	98.4	24.0
0.22	C	7 500 000	105	98.1	25.0
0.22	D	10 000 000	110	98.0	25.0
0.22	E	16 000 000	280	95.0	38.0

* *Journal*, New England Water-Works Association, Vol. 24, p. 589.

Mr. Hazen. It will be seen that the number of bacteria passing increases rapidly with the rate, and whether the total number of bacteria is considered or the *B. coli* results, the number passing is approximately in proportion to the rate. In other words, doubling the rate substanstially doubles the number of bacteria in the effluent.

This is entirely in harmony with all the Lawrence experimental results extending over a period of 20 years. There have been occasional apparent exceptions, but, on the whole, experience with Merrimac River water has uniformly been that more bacteria pass as the rates are higher.

The theory sometimes advanced, that the efficiency of filtration is controlled to a certain extent by gelatinous films, and that, as far as thus controlled, is less dependent on rate, would not seem to be borne out by these results. The Merrimac River water, carrying large amounts of organic matter, would certainly seem better adapted to the formation of such films than the clay-bearing Potomac water, comparatively free from organic matter; but it is the Potomac water which seems to show the least influence of rate on efficiency.

The experiments show that turbidity passes more freely at the higher rates with the Potomac water, as has also been found to be the case with other clay-bearing waters.

In the last lines of Table 20 are given cost per million gallons for filtering at various rates. There is no discussion of these figures, and as they differ considerably from those which the writer has been accustomed to use, the calculation in Table 23, made three years ago for a particular case, may be of interest.

TABLE 23.—RELATIVE COST OF FILTERING AT DIFFERENT RATES.

	Nominal rate, in millions of gallons per acre daily:			
	3	5	10	20
Percentage which average yield is of nominal rate	85	80	75	65
Average output per acre, in millions of gallons per day	2.55	4.00	7.5	13.0
Cost of that part of filters per acre dependent on rate	$12 000	$20 000	$40 000	$80 000
Cost of that part of filters per acre not dependent on rate	50 000	50 000	50 000	50 000
Total cost of filters per acre	60 000	70 000	90 000	130 000
Cost per million gallons of capacity	20 600	14 000	9 000	6 500
Cost per million gallons of average daily output	24 400	17 500	12 000	10 000
Capital charges and depreciation at 6% on cost per million gallons	4.00	2.87	1.97	1.64
Operating expenses, the same at all rates	1.00	1.00	1.00	1.00
Total cost of filtering, excluding pumping, storage, and all auxiliaries	5.00	3.87	2.97	2.64
Relative cost	1.29	1.00	0.77	0.68

When the costs of pumping, pure-water reservoirs usually necessary, etc., are taken into account (which add equally to the cost at all rates), the cost of filtering will vary less with the rate than is indicated. Mr. Hazen.

The effect of rate on cost, as calculated in Table 23, and also the percentages of the bacteria of the raw water found in the effluents by the author and by Mr. Clark, are shown on Fig. 10.

Considering all these results together, and also all the other evidence known to the writer bearing on this point, it seems clear that filters are not as sensitive to changes in rate, within reasonable limits, as has been frequently assumed; but, on the other hand, there is usually a substantial increase in the percentage of bacteria passing through a filter with increased rate.

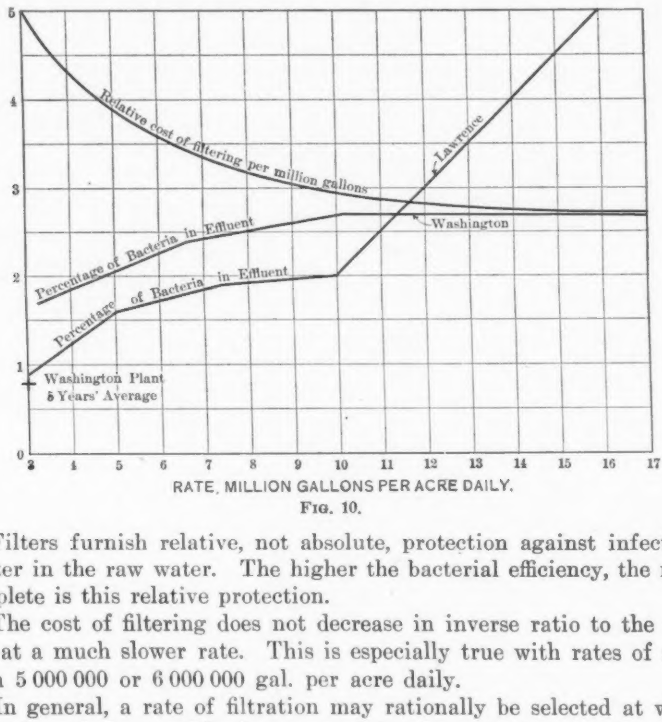


FIG. 10.

Filters furnish relative, not absolute, protection against infectious matter in the raw water. The higher the bacterial efficiency, the more complete is this relative protection.

The cost of filtering does not decrease in inverse ratio to the rate, but at a much slower rate. This is especially true with rates of more than 5 000 000 or 6 000 000 gal. per acre daily.

In general, a rate of filtration may rationally be selected at which the value of the possible danger resulting from an increase in rate is equal to the saving that may be made in cost by its use. This point must be a matter of individual judgment. The tendency of the last few years has been to use higher rates, or, in other words, to cheapen the process and to tolerate a larger proportion of bacteria in the effluent. The use of auxiliary processes has been favorable to this, especially

Mr. Hazen. the use of chloride of lime, in connection with either the raw water or the effluent.

By the judicious use of this substance, efficiency may be maintained while using higher rates than would otherwise have been desirable.

The writer believes that there will be many cases where the added risk of using too high a rate is not worth the relatively small saving in cost that accompanies it.

Mr. Johnson. GEORGE A. JOHNSON, ASSOC. M. AM. SOC. C. E.—This paper contains information of an exceedingly interesting nature. There is comparatively little difficulty in obtaining accurate figures on the cost of construction of water purification works, but, with costs of operation of such works, it is different. The data available in published reports and papers are usually more or less fragmentary, and unexplained local conditions with reference to the character of the raw water, the cost of labor and supplies, and methods of apportioning these costs, introduce variables so wide as frequently to render the published figures almost useless for purposes of comparison.

Mr. Hardy's paper is noteworthy in that it presents certain relatively new features of slow sand filter operation which have been only lightly touched on in water purification literature up to the present time. These refer particularly to means whereby a filter may be continued in service without removing a portion of the surface layer of the filter surface itself when the available head has become exhausted, and to methods whereby washed sand may be expeditiously and more economically restored to the filter than has been the case hitherto.

Sand handling is the most important item of expense in the operation of a slow sand filter. Quite recently a charge of $1.50 per cu. yd. for sand scraping, transportation to sand washers, washing, and restoring to the filter, was not considered exorbitant, but the improved methods developed during recent years at Washington, Philadelphia, Albany, and more recently at Pittsburg (at all of which places hydraulic ejection plays an important part), have shown the feasibility of reducing this figure by nearly, if not quite, two-thirds.

The practice observed at Washington of raking over the surface of the sand layer when the available head becomes exhausted, in order to avoid the cost and loss of time necessitated by shutting down the filter and scraping off the surface layer, is unquestionably one of the most striking advances in slow sand filter operation in recent years. In rapid sand filter operation, to prolong the period of service between washings, agitation of the filter surface has been used to advantage for many years. The full value of surface raking may not be generally appreciated, but the results which have followed a trial of this procedure at Washington, Philadelphia, and Pittsburg have shown that the output of filtered water between scrapings may be doubled or

trebled thereby, with no injury to the filter itself or to the quality of the filtered water. The cost of raking over the surface of a 1-acre slow sand filter unit is less than $10 at all the above-mentioned places, which fact in itself shows the great saving in money and time effected by periodically substituting surface raking for scraping. Under ordinary conditions it has been found that a filter can be raked to advantage at least twice between scrapings. Mr. Johnson.

In the case of filters thus raked, a deeper penetration of suspended matter into the sand layer is inevitable, but at Pittsburg, as at Washington, such penetration does not extend more than about 2 in. below the filter surface. When the filter is finally scraped, a deeper layer is removed, of course, but it is clearly more economical to remove a deep layer at one operation than to remove separately several thinner layers of an equal total thickness.

The lost-time element is an important one, and at Washington this was the main reason for trying surface raking. It became necessary to increase the output of the filters, and the ordinary scraping consumed so much time that the sand-handling force was increased, working day and night. The raking expedient introduced at this time overcame this, and Mr. Hardy states that it is still followed when the work is at all pressing. The speaker has found at Pittsburg, as Mr. Hardy has found at Washington, that raking is nearly if not quite as effective as scraping in restoring the filter capacity.

Eleven years ago the speaker was connected with the preliminary investigations into the best methods of purifying the Potomac River water for Washington. It then appeared that while for the greater part of the time during an average year the Potomac River could be classed among the clear waters of the East, there were periods when excessive turbidity made it necessary to consider carefully methods of preparatory treatment before this water could be filtered effectively and economically. As Mr. Hardy has said, considerable prejudice existed against the use of a coagulating chemical, and the expedient was therefore adopted of giving the water a long period of sedimentation in order to remove enough of the suspended matter to allow the clarified water to be treated on slow sand filters. The expert commission, consisting of Messrs. Hering, Fuller, and Hazen, recommended the occasional use of a coagulating chemical, but this recommendation was not carried out.

The Potomac River is somewhat peculiar, in that the turbidity of its waters, as shown by the results presented in Mr. Hardy's paper, ranges from 3 000 to practically nothing. The bacterial content also varies widely, and Mr. Hardy's tables show this variation to be from 76 000 to 325 per cu. cm. Such a water as this requires particularly careful preparatory treatment. The Dalecarlia Reservoir has a capacity of something like 2 days' storage, the Georgetown Reservoir the same,

Mr. Johnson.

and the McMillan Park Reservoir nearly 3 days, making a total sedimentation of more than 7 days. Without the use of a coagulant, it is significant that during a period of five years, even with 7 days' sedimentation, the average maximum turbidity of the water delivered to the filters was 106 parts per million, and the maximum average turbidity in one month was 250 parts per million. The water filtration engineer can readily understand that waters as turbid as this cannot be treated economically and efficiently in slow sand filters. It would appear that coagulating works might advantageously have been installed at the entrance to the Dalecarlia Reservoir. If this had been done, and coagulant had been added to the water at times when it was excessively turbid, a considerably shorter period of subsequent sedimentation than now exists would in all probability have rendered the water at all times amenable to efficient and economical slow sand filter treatment.

The prejudice in Washington against the use of coagulants has also manifested itself in other localities, but the results which have been obtained during the past twenty years from rapid sand filters and from slow sand filters, treating waters previously coagulated with salts of iron or alumina, have shown how thoroughly unreasonable were these objections. In this connection it is interesting to note that there are in the United States more than 350 rapid sand filter plants, and that nearly 12% of the urban population of Continental United States is being supplied with water filtered through rapid sand filters, in connection with all of which a coagulating chemical is used in the preparatory treatment.

Attention has repeatedly been called to the fact that the relatively high typhoid death rate in Washington, since the filter plant was installed, was a possible indication that the filters were inefficient. It is true that there has not been the marked reduction in the typhoid death rate in Washington, following the installation of the water filtration works, that has been observed in other cities in America. For the six years prior to the date on which filtered water was supplied to the citizens of Washington, the average typhoid fever death rate was 59 per 100 000 population, as against 37 per 100 000 for the five years following, a reduction of 37 per cent. At Albany, N. Y., where the first modern slow sand filter was built in 1899, the typhoid death rate has been reduced by 75 per cent. At Cincinnati, Ohio, the average death rate from typhoid ranged around 50 per 100 000 for years, but since the installation of the filtration plant it has been reduced to a point which places that city, with respect to freedom from typhoid fever, at the head of all the large cities in America; in 1910 the death rate from typhoid in Cincinnati was 6 per 100 000. Similarly, at Columbus, Ohio, where the typhoid death rate before the installation of the filtration plant in 1906 was even higher than at Cincinnati, it was reduced to less than 13 per 100 000 in 1910, whereas, for the previous five years,

it was 61 per 100 000. Philadelphia, before the installation of the filtration works, had a typhoid death rate of 60 or more per 100 000, and in 1910 the death rate from this disease was 17. Pittsburg, at least that part of it now supplied with filtered water, for years had a typhoid death rate of more than 130 per 100 000, but the present rate is about 12 per 100 000. Mr. Johnson.

TABLE 24.—TYPHOID FEVER DEATH RATES IN CITIES OF THE UNITED STATES WITH POPULATIONS IN 1910 OF 100 000, OR MORE.

Statistics gathered by correspondence and from Reports of the Bureau of the Census, Department of Commerce and Labor, Mortality Statistics.

NOTE.—Statistics from Birmingham, Ala., Dayton, Ohio, Fall River, Mass., Louisville, Ky., Memphis, Tenn., Oakland, Cal., and Providence, R. I., are not included, as they are incomplete.

City.	Typhoid Fever Death Rate per 100 000 Population.							
	1906.	1907.	1908.	1909.	1910.	Average for six years, 1900–05, inclusive.	Average for five years, 1906–10, inclusive.	Average for 11 years, 1900–10, inclusive.
Albany, N. Y	20	20	11	19	15	25	17	21
Atlanta, Ga	50	64	47	44	43	65	50	58
Baltimore, Md	34	41	31	23	41	36	34	35
Boston, Mass	22	10	26	14	11	23	16	20
Bridgeport, Conn	10	13	13	13	9	15	12	14
Buffalo, N. Y	24	29	21	23	20	29	23	26
Cambridge, Mass	18	10	10	9	12	18	12	15
Chicago, Ill	18	18	15	12	14	27	16	22
Cincinnati, Ohio	71	46	19	13	6	54	31	44
Cleveland, Ohio	20	19	13	12	19	51	17	36
Columbus, Ohio	45	38	110	17	13	61	45	54
Denver, Colo	68	67	58	24	30	37	49	42
Detroit, Mich	22	28	22	19	16	17	22	19
Grand Rapids, Mich	39	30	30	17	27	34	28	31
Indianapolis, Ind	39	29	26	22	31	76	30	55
Jersey City, N. J	20	14	10	8	10	19	12	16
Kansas City, Mo	38	40	35	23	38	48	35	42
Los Angeles, Cal	18	23	19	18	12	35	18	27
Lowell, Mass	7	9	24	11	21	19	14	17
Milwaukee, Wis	31	26	17	21	45	19	28	23
Minneapolis, Minn	33	26	18	20	58	38	29	34
Nashville, Tenn	66	85	62	53	48	54	58	56
Newark, N. J	18	24	12	11	13	17	16	17
New Haven, Conn	54	30	34	20	17	44	31	38
New York, N. Y	15	17	12	12	12	19	14	17
New Orleans, La	30	56	31	25	28	40	34	37
Omaha, Nebr	28	24	22	31	75	20	36	27
Paterson, N. J	4	11	10	5	7	25	7	17
Philadelphia, Pa	74	60	36	22	17	47	42	45
Pittsburg, Pa	141	135	53*	13*	12*	132	71	104
Richmond, Va	44	41	50	24	22	66	36	53
Rochester, N. Y	17	16	12	9	13	15	13	14
St. Louis, Mo	18	16	15	15	14	33	16	25
St. Paul, Minn	21	17	12	20	20	14	18	16
San Francisco, Cal	..	57	27	17	15	20	29	24 (?)
Scranton, Pa	11	76	11	11	14	18	35	26
Syracuse, N. Y	10	16	15	12	30	14	17	15
Toledo, Ohio	45	36	40	31	32	36	37	36
Worcester, Mass	12	14	10	8	16	17	12	15
Washington, D. C	52	36	39	33	23	59	37	49

* Filtered water section for Allegheny District not included.

Mr. Johnson.

TABLE 25.—AVERAGE MONTHLY RESULTS FOR THE PERIOD, 1905-1910.

Reservoirs.	Period of sedimentation, in days.	Turbidity, in parts per million.	Bacteria per cubic centimeter.	PERCENTAGE REMOVED.	
				Turbidity.	Bacteria.
River	....	106	6 400	..	..
Dalecarlia	2.2	50	5 000	53	22
Georgetown	2.2	38	3 400	24	32
McMillan	2.8	26	2 000	31	41
Totals and averages	7.2	....	......	75	69

While it may perhaps seem unreasonable to single out Washington as a particular sufferer in this respect, it is highly probable that a large share of the typhoid is still caused by secondary infection, flies, impure milk, and private and public wells. The speaker remembers distinctly that ten years ago, when he made an investigation into the purity of the water of about 100 public wells in that city, a large number of them showed unmistakable evidence of being polluted with sewagic matter. Conclusive evidence would be secured to dispel any doubt as to the sanitary quality of the filtered product if hypochlorite of lime were added to the filtered water throughout one year or throughout the typhoid months. It seems strange to the speaker, that for this, if for no other reason, this safe and non-injurious germicide has not as yet been used at Washington, in view of the fact that at the present time it is being used continuously or intermittently in the treatment of the water supplies of scores of the most important cities of this country, among which may be mentioned New York, Philadelphia, Cincinnati, Pittsburg, St. Louis, and Minneapolis.

Mr. Knowles.

MORRIS KNOWLES, M. AM. SOC. C. E. (by letter).—This description of the operation of the Washington Filtration Works is timely and of great interest. It is ten years since the writer, in collaboration with Charles Gilman Hyde, M. Am. Soc. C. E., presented a similar record for the Lawrence, Mass., filter. That paper was the first complete, detailed, and continuous history of the actions and results obtained for a long period of time with such a purification works.* Since then, the art of filtration has advanced in many ways, particularly in regard to the methods of cleaning slow sand filters and in the accompanying processes. It is well, therefore, again to take account of stock and see really what progress has been made. Therefore, Mr. Hardy's paper, giving a description of the operations of a system thoughtfully designed, after long consideration of the problem, and of operations carried on under efficient and economical administration,

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 258.

with thorough record of all details, should furnish a groundwork for the careful consideration of the question stated above. Mr. Knowles.

The writer, using as a text some of the ideas given in the paper, but more particularly some of those becoming prevalent elsewhere, desires to discuss methods and costs of operation, especially in relation to sand handling; and to offer suggestions looking toward greater efficiency, as well as economy, in carrying out the standard and well-tried methods.

Theory of Slow Sand Filtration.—First, what is the process of slow sand filtration? The answer to this question involves many factors, some of which are even yet but imperfectly understood. In the early history of filtration, at the time of the construction of the London filters, only the straining capacity of the sand bed, to remove gross particles, was known. Later, when the organic contents of water had become better understood, the chemical or oxidizing powers of the process were recognized as performing an important part. Finally, co-existent with the discovery of the so-called "germ theory of disease," a study of the bacterial action of filters resulted in the recognition of its importance. It is now universally thought that each of these factors performs its useful function; that the size of the sand, the amount of organic matter remaining on the surface of the bed, the turbidity of the applied water, and the bacterial content of the influent, are some of the things on which depends the determination of the relative importance of each process.

Engineers have been taught to believe, by the German school of thought, that the film of organic matter on the surface of the sand plays a very important rôle in filtration. This *Schmutzdecke,* as it is called, has been considered so precious that stress has been placed on treating it with great care. It was not to be wholly removed at the time of cleaning, and it was not to be walked on, or indented, or in any other way consolidated or destroyed. In fact, in some cases, the wasting of the first water after cleaning has been advocated, for the reason that not a sufficient amount of this organic film would be left on top of the sand to begin the filtration process properly immediately after the cleaning.

In late years, however, there has been a tendency to depart from this fundamental doctrine of slow sand filtration. Various new processes for cleaning the sand surface have been advocated; some of these partly destroy and others completely exterminate any semblance of a bacterial film on the sand bed. These ideas, advanced without any real and serious discussion of their intrinsic merits, or their effects on the public health, are not founded on long continuous records of such results as are necessary to establish confidence in the final value of any of these methods.

Rapid advances along this line have been made more recently,

Mr. Knowles. notwithstanding the occurrence of notable instances of trouble and the resultant need of complete repair of filtration beds. Because of the rough treatment of the sand surface, a penetration of organic matter and filth into the bed had taken place. This caused deep clogging, prevented the usual yield of water, and brought about a lessened bacterial efficiency, due to the attempt to force water through the filters, and because some organic matter and growths in the lower part of the bed had furnished a breeding place for more bacteria.

All these endeavors to reduce the work of cleaning have been commendable, because scraping and sand handling are the items of greatest expense in slow sand filter maintenance. Every one has been desirous of minimizing this cost. However, as the writer will endeavor to show, it seems that attempts along this line should be with the idea of doing more economically, as well as efficiently, the things which one knows will accomplish the proper results, rather than unwisely to adopt new methods which have not been tried for a long enough period to determine their effect on the public health.

Pittsburg Methods.—When first taking up the problem of design in Pittsburg, in 1902, the writer had presented to him for consideration and adoption, a suggestion that a certain method of cleaning sand filters, which would involve the washing of the sand in place (similar to that recently tried at the Jerome Park Experiment Station, New York City), would be advisable and economical. The decision then made has never been regretted. As this plan involved such a complete departure from those principles which had been well tried and had proven successful, it was believed that it was not safe to adopt such a method on the municipal filtration works, from which the people were to derive their drinking water. There is more to be considered in such a problem than mere economy of operation; the economy of human life, the effect on which requires far longer than a few months of trial to determine, is a much more important factor. Believing that no one should depart, until after a long period of conclusive experimentation, from that principle which is known to be safe (viz., to take off a small portion of the clogging surface), the writer studied to determine more efficient and economical methods of accomplishing this end.

A device for scraping the material, in just the same way as with shovels, but more efficiently and more exactly, was developed by George P. Baldwin, M. Am. Soc. C. E., under the general supervision of the Bureau of Filtration, of which the writer was in charge. However, on account of the unfortunate and earlier arrangement of other constructive matters, which the City's Legal Department advised could not be changed without upsetting the contract, the entrance doors to the original forty-six filters were not built large enough to permit the rapid and economical transfer of these machines, and, as this

act takes so large a proportion of the total time of operation, it has not been found economical to use them. The additional ten filters, recently constructed, with doors especially designed and large enough to pass the machines, have not yet been placed in operation. This is said to be on account of lack of funds and of employees. Therefore, there has been no opportunity to demonstrate what the scraping machines can do, under the conditions for which they were designed to operate. The restoring machine, a complementary device in mechanical operation, which simply replaces the sand in the same way that it would be if wheeled back, with a small percentage of moisture, has accomplished its purpose well and economically. The sand is placed in the filters so that there is no further settling; with a smooth surface, needing no additional adjustment; with absolutely no possibility of sub-surface clogging; and with the filters starting off exceedingly well in operative results. Mr. Knowles.

Washington Methods.—In Washington, it is stated that the filters are still cleaned by the old-fashioned method of scraping with shovels, throwing the sand into piles, and afterward removing it with a movable ejector. Between scrapings there is also an occasional mid-period action of raking the unwatered sand surface, for the purpose of stirring up the dirty film. This process does not remove any of the clogging material from the bed, but it is said that no injurious effects are produced, and that it is economical. It is stated that the so-called "Brooklyn method," of stirring the surface of the sand while the water is on the bed, has been tried at Washington, but with unsatisfactory results. It seems to have been advocated with greater fervor in some other places.

The method of dry raking does not remove the dirty material, but loosens up the pores of the surface, and through this porosity permits clogging to penetrate deeper into the filter. The method of raking with water on the bed, although it removes some of the organic dirt, also permits deeper penetration of the remainder. The latest devised system of washing the sand in place, called the "Blaisdell method," thoroughly destroys the *Schmutzdecke* above, and, at the same time, must permit the formation of a subsidiary one below. In the Nichols method, the material removed by scraping is conveyed by an ejector to a portable separator where it receives a single washing; the dirty water overflows to the sewer, while the washed sand is discharged through a hose and deposited on the recently scraped surface. As the latter is partly impregnated with impurities, there is, by this process, a tendency toward sub-surface clogging. All these processes are marked and serious departures from the well-tried method of cleaning slow sand filters, which, it is well known, will operate successfully to purify polluted river waters and make them safe to drink. In all there is the danger that they have not been sufficiently and carefully tried,

Mr. Knowles. under scientific observation, as to results and possible effects on the public health, to be sure that the bacterial efficiency can long continue to be satisfactory, with the application of specifically infected waters. It is dangerous, and may even jeopardize the safety of human lives, to experiment on water which is furnished for drinking purposes. There is also the added danger, well known from past experience, that in a few years (it may be more or less, depending on the extent and intensity of the new workings) the filters will need renovation, partly, if not wholly, throughout the entire bed. Thus, considering the total cost during a long term of years, the apparently cheaper method may become the most expensive.

There is also an interesting query in regard to the Washington method of replacing sand in the filters, and it is worthy of most careful thought and attention. If the process described can be carried on with success and safety, it will prove to be a long and progressive step in the methods of operation. The difficulty, however, is in determining from any short-term runs whether such a process can be continued permanently without impairing the efficiency of the sand bed. Apparently good conditions may change, after a few years' trial, and be followed by unsafe results and predicaments. This replacing of sand with whatever dirt and detritus may travel with it in the carrying water is certainly not equivalent to the care with which it has been understood that sand should be deposited in filters. It is not comparable with the care with which it is placed, when wheeled from a washer, where dirty water overflows the lip, or where it is placed by a machine restorer in the filter, where the transporting water also overflows the weir and is carried to the sewer.

These cheap and rapid methods of doing the work, advanced in the interests of economy, and the idea that sand filters, receiving polluting waters, can operate at higher rates than those which we have demonstrated, and, therefore, have been led to believe are safe, is a speeding up of the whole organization and of operating conditions. It is like speeding up a machine for the purpose of getting a greater output, with the usual result that fast running means quicker wearing out of both man and machine. Quicker operations generally mean carelessness in doing the work, especially in municipal service. Carelessness is engendered by the thought that such work can be handled in a rough and rapid way, and, further, by the ridicule of all these things, which we have learned to be careful about, as old-fogyish, out-of-fashion, and archaic. Carelessness in operation breeds contempt for the art. Some of the less efficient filter plants, from the standpoint of effect on the public health, may reflect such ill-considered methods.

Economy with Efficiency in Operation.—It is particularly important to find out whether one can secure the desired economy, and, at the same time, the required efficiency. The development of efficiency in every line of human endeavor is receiving much attention at present,

Mr. Knowles.

and not the least cause for this is the growing recognition of the demand for a high standard of service for the rate charged. One of the first requirements is to have well-defined ideals and standards. When one knows how to secure a good and safe result, it is unwise to depart therefrom for a mere whim, or to secure a supposedly lessened expense, unless other facts be also determined favorably. The desire for economy must be tempered by good sense, which means that one should be willing to change a method only when the wisdom of such has been clearly demonstrated. Efficient service can only be secured by good discipline, accompanied by fair dealing. This means employing no more men than are actually necessary, paying them on the basis of the standard of service and output produced, taking an interest in the working conditions, and providing for their health and welfare.

About twelve years ago, the writer made some investigations of the efficiency of laboring gangs in scraping and handling sand at filter beds,* and found that ten men was the most economical number to use in scraping the surface of the Lawrence filter, as then built and operated. This result was determined by numerous studies of the output per man per minute, with different numbers of men working under different conditions. This same sort of study has been carried further by adepts in the art, in reference to shop and similar management, but one fails to find corresponding development along this line in municipal organization except by a few of the scattered Bureaus of Municipal Research. These results, also, have related to a few of the more common operations, such as determining the cost per mile, or per square yard, of street cleaned, or per million gallons of water pumped.

The cost of the management of water-works, one of the largest factors of public enterprise, has never been investigated extensively and thoroughly. There is much possibility in planning for greater efficiency and in determining what can be accomplished under economical administration. Every one is aware of the multiplicity of men in municipal service. Some of these are entirely incompetent, others partly so; the recent appointees may be more efficient, but the majority of them gradually deteriorate under the subtle influence of the prevailing atmosphere, and each new incoming administration places more and more men on the work, without reason or necessity. All these tendencies have made the cost and maintenance of public work greater and greater, and, at the same time, have resulted in frequently and steadily decreasing the output and efficiency per employee.

The Washington situation, however, presents an admirable contrast to this, because of the methods of administration of the public works of the District of Columbia and their freedom from petty political influence. The limited number of employees has tended toward economy, and rendered this plant the envy of all who have desired to

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 291.

Mr. Knowles. obtain good management. Its cost items have been looked on as a result long hoped for, but seldom obtained. It is to be regretted, therefore, that such an abrupt change in methods of removing clogging material and replacing sand has taken place without years of experimental trial on filters not furnishing drinking water to the public, and without an attempt, under such excellent conditions, to maintain the efficiency by a better labor output and by improved working and machine methods in the performance of the older and established order of doing things.

In preparing water for the use of the people, the realms of the unknown are so much larger than those which have been investigated and developed that there may be many factors affecting the public health, and many ways in which it is dangerous to depart from well-known and surely safe methods. Who can say that in some subtle and, at present, unknown manner, the failure in some places, where filtration is practiced, to reduce the death rate from typhoid fever may not be due to the introduction of radical departures from the older, slower, safer, and more efficient methods which have produced such excellent results, both in America and in Europe? Further, in cases where there has been a falling off in the typhoid death rate, the failure to secure an accompanying improvement in general health conditions, which follows so closely in communities supplied by water filtered in accordance with the more conservative principles, may be due to the introduction of some of these not thoroughly tried processes. Some day full information may be available as to the influence of these methods of plant operation on the health of the community. Until that time, is it not a much better policy to follow the principles which have been proven by many years of experience to produce safe results, and to make the foremost object the improvement of the methods of operation in accordance with these established truths?

There is opportunity for the upbuilding of greater efficiency in the conduct of employees and in securing the maximum output, because of more comfortable and healthful conditions than usually exist. The elimination of political influence from municipal service is a task which challenges the people of to-day. The operating and managing engineer is in a position to perform an important part in accomplishing this end. The number of employees can be reduced to those actually needed, and the way opened for the employment of men who thoroughly understand the necessities of honesty and efficiency in the conduct of public affairs. It should be remembered that to design and construct well is only half the job; to operate economically and efficiently is even more than to build, and requires just as good talent, just as keen appreciation of the various problems, and is even more essential to public welfare. It seems to the writer that the logical development of the art of obtaining economy as well as efficiency should be along these lines,

rather than to revolutionize methods without having a long-period test of their value, and at the same time allow political influences to control, to a large extent, the labor item. Mr. Knowles.

Preliminary Treatment.—The decision as to the preliminary treatment of the Potomac River water before filtration is of interest, particularly because various other decisions have been reached in different sections of the country. However, in the main, these decisions have been due to differences in the character of the waters, but it must be evident that they have sometimes been the result of ill-considered action, or the desire to promote some special interest. The use of preliminary filters, which involves a large investment, is not always to be commended, particularly because at times of reasonably good water the removal of some of the organic matter is really injurious and lessens the effect of the final filters.

For a long time, the writer has believed that, where other things are equal, and where there is no important reason for double or preliminary filtration, long periods of storage, accompanied by the use of coagulant at times of severe and extreme muddiness, as planned at Washington, solves the problem in the most practical and economical way. It is true that the investment for a large storage basin may equal, or even exceed, that required for preliminary filters; but the influence of storage on the quality of raw water is never injurious, and, by ripening the condition of the water, may be greatly beneficial in the process of filtration.

The storage available in such a basin makes it possible to shut off the supply from the river during the worst conditions of the water. The duration of the most troublesome spells ordinarily does not exceed a few days, and it is usually possible to secure sufficient capacity in the basin to tide over these periods. Then again, long periods of storage, in addition to assisting in breaking up organic matter, permit the dying out of bacteria, particularly many of the pathogenic kind, and, therefore, the water is rendered much safer from this standpoint. In other words, there is additional insurance in long storage against the faulty and careless operation of incompetent filter employees. The addition of coagulant, especially the fact that only a very small investment of capital is required for the necessary apparatus for dosing the water, and that the cost of the coagulating materials has to be met only when used, seems to give the process, in a most satisfactory manner, the requirement for economical management and thoroughness in preparing the water for final filtration.

Parking Public Works.—It is disappointing that the author has not mentioned some of the steps contemplated in reference to the landscape treatment of the Washington filtration area. Probably every one has been impressed by the barren aspect of the works as they are approached, and as one looks over them. Recently, however, it is

Mr. Knowles. stated that some steps have been taken to lay out the grounds, treat the surface in an attractive manner, and make a park of the area. The writer has a firm opinion that when an investment is made for public works, it costs but little in addition to construct buildings along appropriate architectural lines, to treat the grounds in a pleasing manner, and to make the entire works a credit to the municipality from an artistic standpoint. When treated on broad lines, such areas become public parks, and afford open breathing places for the residents, and, if near centers of population, may well be equipped with playground facilities for the children.

The influence which these ever-present examples of attractiveness have on the community is becoming better recognized by students of social progress, and there seems to be no doubt that spending money on such features is not only desirable from the artistic standpoint, but is justified on practical grounds as well. It is cheaper than to create parks, when necessity and demand can no longer be resisted, by buying property and occasionally tearing down buildings and constructing *de novo*. That this work is now being done in Washington, even after construction, is certainly a recognition of the advisability of efforts in this direction.

Mr. Whipple. GEORGE C. WHIPPLE, M. AM. SOC. C. E. (by letter).—Mr. Hardy's paper is an excellent presentation of the results of the operation of the Washington water filtration plant from the time of its construction in 1905 until June, 1910. Papers of this character are altogether too infrequent, and the actual results from the filters now in use are not readily accessible in detailed form. Yet it is only by studying the results obtained by filters in actual use that improvements can be made and the art advanced.

Among the many important facts brought out by Mr. Hardy, only a few can be selected for discussion. One of these is the operation of filters under winter conditions. It is well known that the efficiency of sedimentation basins and filters is lower during winter than at other times, yet it is just at this season of the year that there is the greatest danger of typhoid fever and similar water-borne diseases being transmitted by water. Most of the great typhoid epidemics have occurred during cold weather, and the very use of the term "winter cholera" is of significance. Apparently, typhoid bacilli and similar bacteria are capable of living and retaining their vitality longest during that season of the year. Just why this is so, bacteriologists have not satisfactorily explained. Doubtless many factors are involved. Because of the increased viscosity of the water, sedimentation takes place less readily at lower temperatures, and inasmuch as sand filtration is partly dependent on sedimentation, the efficiency tends to fall off in cold weather. During winter some of the external destroying agencies

are less potent, such as the sterilizing effect of sunlight, and the presence and activity of some of the larger forms of microscopic organisms which prey on the bacteria. Another factor may be the greater amount of dissolved oxygen normally present in water during cold weather, as experiments have shown that dissolved oxygen favors longevity. Mr. Whipple.

Still another reason for the larger numbers of bacteria that pass through a water filter during cold weather may be the effect that the low temperature has on the size of the bacteria themselves. A few experiments made recently by the writer appear to indicate that at low temperatures the gelatinous membrane which surrounds the bacterial cells tends to become somewhat contracted, thus decreasing the apparent size of the bacteria as seen under the microscope. Either this contraction occurs, or the cells themselves are smaller when they develop in the cold. It is possible also that low temperature affects the flagella of the organisms in the same way. It is not unreasonable to suppose that the effect of low temperature is to form what may be, in effect, a protective coating around the cells, which tends to make them smaller, less sticky, and less subject to outside influences. This would tend to make them pass through a filter more readily. In line with this idea also is the well-known fact that disinfection is less efficient in cold water than in warm water.

Another way of viewing the matter is that cold retards the growth of bacteria on the filter, thus reducing the effect of the *Schmutzdecke*. Still another view of the greater danger from bacterial contamination in winter is the theory that cold prolongs the life of the bacteria by merely preventing them from living through their life cycle and reaching natural old age and death as rapidly as in warm weather.

Another topic in Mr. Hardy's paper which has interested the writer is that of preliminary filters. The experiments described at length indicate clearly that such devices would prove of little or no benefit under the conditions existing in Washington, and that when the river contains considerable amounts of suspended clay nothing less than chemical coagulation will suffice to treat the water so that the effluent will be perfectly clear. Preliminary filters have been used for a number of years at various places and with varying success. In few instances have they been operated for a sufficient length of time or been studied with sufficient care to determine fully their economy and efficiency as compared with other possible methods of preliminary treatment.

Among other experiments on this matter are those made at Albany, N. Y., and published by Wallace Greenalch, Assoc. M. Am. Soc. C. E., in the Fifty-ninth Annual Report of the Bureau of Water for the year ending September 30th, 1909. The Hudson River water used at

Mr. Whipple.

Albany is quite different in character from the Potomac River water used at Washington, as it is less turbid and contains rather more organic matter. The results obtained in these experiments showed that during the summer the number of bacteria in the effluent from the experimental sand filter used in connection with a preliminary filter did not differ widely from the number found in the effluent of the city filter where there was no other preliminary treatment than sedimentation. In the winter, however, the numbers of bacteria did not increase in the effluent from the experimental filter as they did in the effluent from the city filter. This is shown by Table 26, taken from the report mentioned.

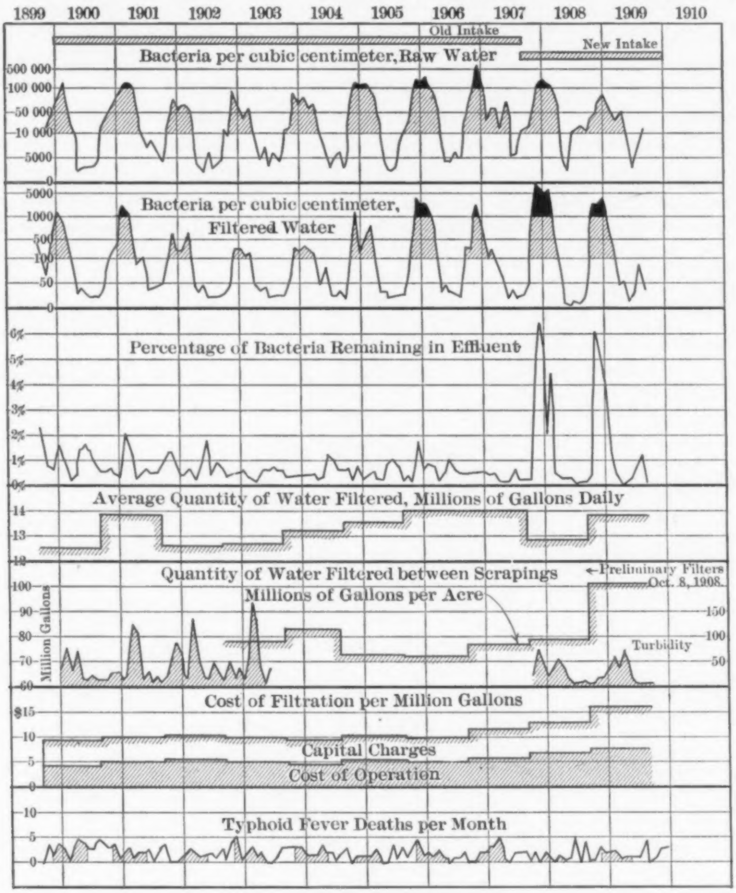
TABLE 26.—Results of Experiments with Preliminary Filter at Albany, N. Y.

Month, 1906.	Bacteria in raw water.	Bacteria in preliminary filter effluent.	Bacteria in effluent from experimental sand filter.	Bacteria in effluent from city filter.
March	133 480	36 000	151	706
April	77 420	4 810	72	155
May	15 800	2 250	48	37
June	4 520	358	38	34
July	2 090	163	25	22
August	2 740	121	36	22
September	8 280	445	20	24
October	38 350	4 235	67	227
November	67 910	15 570	337	341
December	645 500	25 440	144	2 783
1907.				
January	127 560	4 660	48	443
February	28 000	1 800	13	116

Apparently, therefore, at Albany the benefits of the preliminary filter, as far as bacterial efficiency is concerned, would be confined to a short period of three or four months in each year. Under such circumstances it may well be questioned whether the advantages of preliminary filtration justify its cost.

On the diagram, Fig. 10, will be found various data taken from the published records of the Albany filter, from 1899 to 1909. These data include: The numbers of bacteria before and after filtration; the percentage of bacteria remaining in the effluent; the average quantity of water filtered, in millions of gallons per day; the quantities of water filtered between scrapings; the turbidity of the raw water; the cost of filtration, including capital charges and cost of operation; and the typhoid death rates of the city per month. Several points are brought out conspicuously by this diagram. One is the uniformly low death rate from typhoid throughout the entire period. The filter was operated from 1899 until the fall of 1907 with raw water taken from what is known as the "Back Channel." Since then it has been taken

from a new intake which extends into the Hudson River itself. Mr. Whipple. Until the fall of 1908 the preliminary treatment consisted merely of sedimentation, but since then the water has received an additional preliminary treatment in mechanical filters operated without coagulant,



FILTERS AT ALBANY, N.Y.
RESULTS OF OPERATION,
1899-1909.
Compiled from data in Annual Reports
FIG. 11.

along the lines of the experiments just mentioned. During this time the average rate of filtration of the sand filter has not changed materially, although it is said that the maximum rate has been increased since the preliminary filters were put in service. The study

Mr. Whipple.

of the bacteriological analyses shows that the best results were obtained during 1902, 1903, and 1904. Since then the numbers of bacteria in both the raw and filtered water have increased. This was especially noticeable during the winters of 1907 and 1908 when the water was taken from the new intake. It will be interesting to compare the results after the preliminary filters have been operated for a long enough period to ascertain their normal effect on efficiency and on the increased yield.

Another fact to be drawn from the plotted Albany data is the increase in the cost of filtration, both in capital charges and in operation. From 1899 until 1906 the cost of operation, including the cost of low-lift pumping, was approximately $5 per million gallons of water filtered; and the total cost of filtration, including capital charges, was about $10 per million gallons. During the year ending September 30th, 1909, the cost of operation had increased to $7.63 per million gallons, and the total cost of filtration to $15.92 per million gallons, or approximately 50% in three years.

TABLE 27.—Results of Bacteriological Analyses of Samples of Water at Peekskill, N. Y., Before and After Filtration.

Bacteria per cubic centimeter.

Date.	Raw water.	Clear reservoir.	Effluent No. 1.	Effluent No. 2.	Effluent No. 3.	Effluent No. 4.	Tap in city.
1909.							
December 29th	190	100	..	..	..	..	..
1910.							
February 15th	135	10	10	30	20	..	265
March 31st	225	50	25	45	60	..	35
May 18th	300	29	22	26	35	43	36
July 6th	300	44	9	3	41	10	31
August 16th	60	5	0	4	1	13	15
October 3d	550	14	12	14	38	..	..
November 21st	315	22	26	17	6	..	..
1911.							
January 25th	415	7	8	4	6	..	7
Average	277	30	14	16	26	22	65

Tests for *B. Coli.*

Quantity of water tested.	Percentage of Samples Containing *B. Coli.*	
	Raw.	Filtered.
0.1 cu. cm.	0	0
1.0 cu. cm.	20	0
10.0 cu. cm.	40	0

Mr. Whipple.

As a matter of record, the results of a series of analyses made at Peekskill, N. Y., during 1910 are presented in Table 27. A sand filter was constructed for the water supply of this city in 1909, and put in operation in December. The filter has a capacity of 4 000 000 gal. per day. The supply is taken from Peekskill Creek, and the water receives about one week's nominal storage before flowing to the filters. An aerator is used before filtration during the summer, when algæ are likely to develop in the reservoir. The filter was installed after an epidemic of typhoid which was apparently caused by an infection of the water supply. Normally, the water has been little contaminated, but the supply is subject to accidental contamination at any time, among other possible sources of infection being the camps of workmen now engaged in constructing the Catskill Aqueduct for New York City.

TABLE 28.—Average Results of Chemical Analyses at Peekskill, N. Y., Made at Intervals of Six Weeks During 1910.

	Parts per Million.			Parts per Million.	
	Raw water.	Filtered water.		Raw water.	Filtered water.
Turbidity	2.	0	Total residue	70.	76.00
Color	25.	20.	Loss on ignition	19.00	17.00
			Fixed residue	50.00	59.00
Nitrogen as albuminoid ammonia	0.112	0.076	Iron	0.17	0.13
			Total hardness	38.70	45.10
Nitrogen as free ammonia	0.024	0.006	Alkalinity	33.90	42.60
Nitrogen as nitrites	0.001	0.001	Incrustants	4.60	4.50
Nitrogen as nitrates	0.06	0.06	Chlorine	2.60	2.70

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PAPERS AND DISCUSSIONS

THE PITTSBURG AND LAKE ERIE RAILROAD CANTILEVER BRIDGE OVER THE OHIO RIVER AT BEAVER, PA.

Discussion.*

By Messrs. C. W. Hudson, Henry S. Prichard, and Albert B. Hager.

Mr. Hudson. C. W. Hudson, M. Am. Soc. C. E. (by letter).—The form of truss adopted for this bridge is typical of American practice. It is possible, however, that the form has been too closely adhered to, and that many of its supposed advantages are illusory.

Where the requirements of grade and clearance demand a floor of small depth, the conditions are usually met, as in this case, by some form of subdivided truss. This introduces in the same main panel of the cantilever arm a vertical member, which receives its maximum live load from the floor-beam at its foot, and a horizontal member, which is stressed to the maximum either with or without the load at the foot of the sub-vertical. The sub-vertical cannot be cambered for both conditions of maximum stress in the chord, and, therefore, the chord is subject to considerable transverse load in addition to its direct stress. As they carry only one panel load the sub-verticals are small members and cannot properly hold all parts of the chord against column failure as they are generally assumed to do.

Those sub-members which are not subject to primary stress and are intended only to shorten the unsupported lengths of compression members, sometimes pull or push the latter seriously out of line, and as they are small in section and long in length do not properly accomplish the duty for which they are intended.

*This discussion (of the paper by Albert R. Raymer, M. Am. Soc. C. E., published in *Proceedings* for January, 1911, and presented at the meeting of March 15th, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Hudson.

The size of the sub-verticals, with reference to the main post and main diagonals at the main piers, is clearly shown on Plate V, while the size of the bottom chords is not shown in direct relation to the sub-vertical. It can be stated that while they are large enough to hold the chords in a definite position in a vertical plane, they cannot fully hold them from twisting and failure about their plane of least resistance to flexure. The theory of flexure, and tests on full-sized columns, show that the plane of least resistance to flexure is not in general that of either of the principal axes of the member.

Careful study of the displacements which the panel points of a sub-divided truss must undergo under the action of live load raises serious question as to the usefulness of sub-members, and for trusses of great height and relatively short main-panel length, they may be a source of danger.

It is believed that a long truss should have a long panel length which should be nearly equal to the truss depth at any point, in order that secondary stresses may be small.

There is no question but that the floor may be kept shallow in a long panel, and that long panels may be erected by the use of temporary members.

For short spans, the pin-connected truss with tension members of eye-bars has been driven out of use by a better truss having riveted joints and tension members of such form that their component parts are made to act as a unit and to permit of making suitable connection between the members meeting at a point.

It is believed by many engineers that the riveted truss for long spans is decidedly superior to the eye-bar, pin-connected form, although this has not been completely proven by the results of experience.

This structure may be, and undoubtedly is, safe in every way, but the same result could have been accomplished better by a riveted truss without sub-members.

Mr. Prichard.

HENRY S. PRICHARD, M. AM. SOC. C. E. (by letter).—The Ohio River Railroad Bridge at Beaver, Pa., is one of the great bridges of the world. It is notable not only for its size, capacity, and safety, but for the foresight, care, and skill with which it was designed and constructed; the originality and ingenuity by which some of the problems presented were mastered; and the complete success which rewarded the work of the engineers who planned it and supervised its fabrication and erection.

In view of the importance of the bridge and the ever-increasing loads on American railways, it was wise to provide for a load of 6 000 lb. per lin. ft. of each track, and corresponding engines; and in view of this heavy load and of the daring, not to say hazardous, unit stresses permitted in some large bridges, it is refreshing, and tends to restore confidence in American bridge engineering to have the author refer

Mr. Prichard. to the limiting stresses for combined dead load, live load, and ample impact—16 000 lb. per sq. in. in tension and 14 000 lb. in compression —as high.

The regard for the life and safety of the bridge displayed in the choice of loads and unit stresses was consistently carried into the design of the details and the supervision of the shopwork and erection.

The horizontal girders which would protect the bridge and guard against accident in case of derailment, by compelling the train to skid over the ties instead of bumping into the trusses, and the extra stringers to support the ties in case the train should skid, will probably never be needed, but if they ever should be, they will be very much needed. They constitute an insurance against accident to life and property of a very satisfactory kind which substitutes prevention for compensation and is well worth all it cost.

The radial roller bearings under the towers at the ends of the channel span solve in a very satisfactory way the problem presented by secondary stresses in chords and unequal pressure on masonry by practically avoiding them. This detail was expensive, but its cost was a mere bagatelle in comparison with the total, and it may prolong the life of the bridge many years and, in the meantime, will contribute to its reliability.

In this connection it is well to point out that the enormous load on the channel piers, by reason of the problem presented of designing satisfactory shoes and details, is doubtless responsible for the tall tower posts which divert to themselves a considerable portion of the load which otherwise would travel down the diagonal end posts, as a straight top chord between the ends of the diagonal end posts would have been more economical as regards the weight of the main section of the trusses.

The problem of secondary stresses induced by the endeavor of the floor system to take part of the chord stresses has been avoided by providing for expansion in rail joints and in connections of stringers to floor-beams. This problem is a serious one, in long-span bridges, and is often neglected, with the result that serious horizontal bending stresses are developed in the flanges of the floor-beams, and tension in the rivets connecting the stringers to the floor-beams.

The author calls attention to the fact that:

"The dead-load stresses were computed from weights figured from the stress sheets. This method was repeated until the sections, computed on the basis of the actual dead load, figured from the shop drawings, came within 2% of the sections used."

This careful and conscientious method should be a universal practice in designing long-span bridges (in which the dead load is such a large proportion of the total), and not a matter for special commendation, but prominent cases in which it was not followed and in which

the dead-load stresses were greatly under-estimated, are yet fresh in the memories of engineers. Mr. Prichard.

The study which was given in advance to difficulties likely to arise in building and connecting such massive members, and the efficient manner, of which the author makes mention, in which the inspectors discussed such difficulties in advance with millmen and shopmen, are especially to be commended. It is hoped that the author will describe some of these difficulties and the efficient manner in which they were overcome.

The success achieved in anticipating and providing in advance for the difficulties to be encountered in carrying out the plans, and the precision with which the work went together and maintained its proper levels and alignment during erection, were due to the large range of pertinent talent in the employ of the Railroad and the Contractor, and the broad-mindedness of the Chief Engineer and Assistant Chief Engineer of the Railroad, who exemplified the difference between self-confidence and self-sufficiency by deriving the full benefit of the talent at their command, by keeping in touch and discussing important points with other members of their profession, and by keeping in their own hands the final decision of all important matters.

The bridge marks an advance in bridge engineering, and should be a precedent for other large structures.

Albert B. Hager, Assoc. M. Am. Soc. C. E. (by letter).—There are three features connected with the design and fabrication of this bridge which can well be emphasized, as they may otherwise be lost sight of in the many more impressive facts involved in the execution of this project. The thorough manner in which these three features were taken care of shows clearly that the Railroad, the Consulting Engineer, and the Contractor, all had a proper appreciation of their great importance. Mr. Hager.

These features are:

(1) The care exercised in determining the true dead load from which the dead-load stresses were computed;

(2) The fact that the structure was practically assembled in place at the shop, instead of depending largely on reaming to templets for the accuracy of shopwork and the fit of field connections;

(3) "The fact that the material and the work were in a certain sense inspected before the work was done."

(1).—It would hardly seem necessary to call attention to the importance of determining the true dead loads for long and large structures in which the dead-load stresses form such a large proportion of the total stresses. For these comparatively unusual structures, the

Mr. Hager. data, ordinarily at hand for predicating the dead load with reasonable accuracy, do not exist, and therefore the true dead load must be determined by successive approximations based on the actual design.

It is unfortunately true, however, that some large bridges have been built without checking up the assumed dead load properly, and subsequent investigations have shown that these assumed dead loads were considerably below the actual weight of the completed structures. It will be remembered that the Royal Commission which investigated the causes of the collapse of the Quebec Bridge found that the dead load used in the design under-ran the weight of the finished structure by an amount sufficient to have required the condemnation of the bridge, if completed as designed. The writer has participated in the investigation of the strength of two of our largest bridges. In the first of these the purpose of the investigation was to determine whether the live load could be increased without rendering the structure unsafe, and it was found that this could not be done—not because of the increased live-load stresses, but because the dead-load stresses, computed from the weight of the completed structure, exceeded those used in the original design by nearly one-third. In the second of these investigations, it was found that under-estimating the dead load was one of three contributing causes to the weakness of the structure under the specified live loads.

The dead-load stresses in bridges of unusual size or construction should be re-computed after the completion of the working drawings, and the sections revised if necessary, and no shopwork should be begun until this has been done.

(2).—The practice of reaming field connections to templets, which has been generally used by fabricating companies with ordinary structures, is one which the writer believes to be decreasing, and one which will decrease still more. The efficiency of these templets depends as much on the accuracy of their setting, as on their accuracy of construction, and this requires supervision of a grade not generally to be obtained at a bridge shop.

The inspector cannot check up the fit of field connections, on the finished members, with the same accuracy as is possible in laying out the unassembled material. The cost of assembling structures complete at the shop may be slightly greater than that of reaming the field connections to templets, but the increased certainty of easy erection and freedom from back charges, for correcting shop errors in the field, should go far toward offsetting this. The perfection with which the members of this structure went together in the field is undoubtedly due in large measure to the fact that they had already been assembled in place at the shop. Many, but not all, of our large structures have been thus assembled, and this practice should be made general.

Mr. Hager.

(3).—A completed steel member which has not been properly fabricated is at best a difficult thing to repair so that it will satisfactorily serve its purpose, and very frequently it cannot be repaired. At the same time, these unsatisfactory members represent a greater or less amount of the contractor's money, and their unqualified rejection works a very considerable (even though deserved) hardship on him. The problem of where to draw the line in this question, and when to say that a member can be satisfactorily repaired and when not, is the hardest one that the inspector has to face, but it is one that is continually being presented to him. Inspectors, as a class, are not men of technical education or experience in other departments of the work than their own, and yet are subject to the temptation to decide matters, involving considerations with the importance of which they are not familiar, without referring these questions to the proper authorities, because of the delay which this would involve, and the pressure they are under not to hold up the progress of the work.

For these reasons, the writer believes that the most important part of shop inspection is not, as is frequently assumed, the checking up of the dimensions and connections of the completed material, but the inspection which Mr. Raymer refers to as taking place before the work is done. A careful study, by the inspector, of the methods in vogue at the particular shop in question, and the talking over of the difficult points with the shop superintendent and foremen, will usually secure work of much better quality, and with much less friction, than can otherwise be done. This will also tend to remove the feeling of antagonism which the shopmen frequently have for the inspector.

The same reasoning applies to the inspection of the material at the mills, and its more general application would lead to securing better material and more efficient inspection.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES ARCHBALD, M. Am. Soc. C. E.*

DIED OCTOBER 4TH, 1910.

By the death of James Archbald, this Society loses one of its oldest and most highly esteemed members, and one who will be sadly missed at its annual reunions, at which he was a most constant attendant.

James Archbald was born at Sand Lake, N. Y., the temporary residence of his parents, on February 13th, 1838. He was a son of James and Augusta T. Archbald, and was of mixed Scotch and New England ancestry. His father was born on the Little Cumbrae Isle, off the coast of Ayrshire, Scotland, being descended on his mother's side from the Reverend Robert Wodrow, a prominent Presbyterian divine and writer; and his mother was the daughter of Major Thomas Frothingham, of the well-known family of that name of Charlestown, Mass.

James Archbald, Senior, was connected with the Delaware and Hudson Canal Company for nearly thirty years, its mining operations at Carbondale, Pa., being under his superintendence from a very early date. He was afterward Chief Engineer of the Delaware, Lackawanna and Western Railroad for some fourteen years, being stationed at Scranton, Pa.

Following in the footsteps of his father, Mr. Archbald took up the Engineering Profession. He prepared for college in private schools at Carbondale, Pa., and at Manlius, N. Y. After some little experience in the field, he entered Union College, taking the Engineering course, then under the brilliant direction of Professor Gillespie, and was graduated with high honors in the Class of 1860. In the meantime, in 1857, his family had moved to Scranton, Pa., and there, after his graduation, Mr. Archbald became associated as Civil Engineer with the Delaware, Lackawanna and Western Railroad in the capacity of Assistant to his father. On his father's death, in 1870, he was advanced to the Chief Engineership, and remained with the company in that position until 1899, giving it nearly forty years of service.

The Archbalds—father and son—were thus identified with the development of the two great coal companies of the Northern Anthracite Region, The Delaware and Hudson, and The Delaware, Lackawanna and Western, their joint association with these companies extending over a period of more than seventy years. Among the

* Memoir prepared by C. C. Rose and William M. Marple, Members, Am. Soc. C. E.

engineering achievements in which the older Archbald participated was the change of the Delaware and Hudson, in the middle Fifties, to a gravity road between Carbondale and Honesdale, previous to which time the cars were drawn back and forth, from one plane to the other, on a dead level, by horses; and its extension as a locomotive road from Carbondale in the direction of Scranton. The younger man was engaged on the Delaware, Lackawanna and Western Railroad, in the completion of the Van Ness Gap Tunnel at Oxford, N. J.; the building of the Bergen Tunnel at Hoboken, and the approaches to it across the Hackensack Meadows; and the extension of the railroad from Great Bend, Pa., its previous terminus, first to Binghamton, and then to Utica and Buffalo, with the arrangement of the terminals in the latter city. The construction of the Bergen Tunnel, with its approaches, and the Buffalo extension, were under Mr. Archbald's immediate direction as Chief Engineer, and may be regarded as monuments to his superior skill and energy.

In August, 1862, the darkest period of the Civil War, Mr. Archbald enlisted, and was made Captain of Company I of the One Hundred and Thirty-second Pennsylvania Volunteers, which was recruited from Delaware, Lackawanna and Western Railroad men; and with his company, a month later, he took part in the battle of Antietam, being under fire at one of the most exposed and bloodiest points in that memorable battle. He served until January, 1863, when he applied for and obtained a discharge, being threatened with permanent deafness due to exposure.

In 1883, without severing his relation with the Delaware, Lackawanna and Western Railroad, but being relieved of some of his more active duties, Mr. Archbald became interested in the Barber Asphalt Paving Company, and took charge of its paving work in different sections of the country—Buffalo, New Orleans, St. Louis, Omaha, and Portland. Later, however, he severed this connection and resumed his work with the Delaware, Lackawanna and Western Railroad, continuing with that Company until 1899.

After his retirement, Mr. Archbald gradually gave up his professional activities, but was called at times to important undertakings. In the winter of 1900 he had charge of a survey to connect the various independent collieries in the Lackawanna and Wyoming coal regions, with the Erie and Wyoming Railroad, looking to the building of a new anthracite coal road to tidewater. After the abandonment of that project he laid out a line of railroad across the Allegheny Mountains, in West Virginia, for the Cherry River Lumber Company, to connect with the Chesapeake and Ohio Railroad. He also, in 1902, made surveys for the extension of the West Virginia Central Railroad to tidewater, a project made unnecessary by the purchase of the Western Maryland. His last professional work was as Chief Engineer

of the Mississippi Central Railroad, laying out and superintending the construction of its road across the State of Mississippi, from Natchez to Hattiesburg, with a projected extension to the Gulf at Scranton, Miss., with which he was occupied from 1905 to 1907, retiring finally, at the conclusion of this work, with powers unimpaired, at the age of seventy.

Mr. Archbald was a Member of the Institute of Mining Engineers, and was one of the Founders and the first President of the Scranton Engineers' Club. He was a Director of the Third National Bank of Scranton, from its organization in 1872; a Director of the Scranton Gas and Water Company, a Trustee and Vice-President of the Albright Memorial Library, and a Director of the Pennsylvania Oral School for the instruction of deaf children. He was also, for a number of years, a Director of the Scranton Savings Bank.

On January 25th, 1865, he was married to Miss Maria H. Albright, a daughter of the late Joseph J. Albright, General Sales Agent for the Delaware and Hudson Company, and his widow and six children, Col. James Archbald, of Pottsville, Pa.; Joseph A. Archbald, of Buffalo, N. Y.; Mrs. John C. Kerr, of Englewood, N. J.; Rev. Thomas F. Archbald, Mrs. John H. Brooks, and Miss Ruth S. Archbald, of Scranton, survive him. He is also survived by his youngest brother, United States Judge, Hon. Robert Wodrow Archbald.

In August, 1910, Mr. and Mrs. Archbald went to Europe for a three months' tour. It was purely a pleasure trip, and was to extend through Switzerland, Austria, and Italy. Mr. Archbald was apparently in the best of health and spirits; but it was found that his heart was affected by the high altitudes of the Tyrol, and he went to Vienna, for medical examination, and was declared to have no organic infirmity, but was advised not to exert himself unnecessarily. Journeying on to Venice, he seemed to improve, and spent a week in that city, but just as he was leaving, on October 4th, he was stricken at the station, and died the same evening in a hospital. Thus passed away one who loved and deserved well of the Profession, and who was loved and respected by all who knew him.

Mr. Archbald was genial, unassuming, of simple presence, of sterling integrity, and a hater of shams. He was possessed of the highest engineering ability, especially in the field, which was his school rather than the office. It is not necessary to dwell on his characteristics to those who had the pleasure of his acquaintance; but, for those who had not, let it be recorded, that no better friend, no worthier citizen, no more active and few more able engineers, could be found.

Mr. Archbald was elected a Member of the American Society of Civil Engineers on May 15th, 1872.

JAMES HENRY COVODE, M. Am. Soc. C. E.*

DIED SEPTEMBER 9TH, 1909.

James Henry Covode was born in Westmoreland County, Pennsylvania, on October 18th, 1858. He was a son of the Honorable John Covode, a Representative in Congress for twenty-six years, covering the period of the War of the Rebellion, during which he earned the title of "Honest John Covode" as a Member of the War Committee. He subsequently served as a Member of the Committee to investigate the charges against President Buchanan. Mr. Covode's mother was Margaret Peale, a descendant of Rembrandt Peale, the famous artist.

After preparing at the Chester Military Academy, he was graduated with the degree of Civil Engineer from the Rensselaer Polytechnic Institute in 1882. After graduation, he went to the Pennsylvania Railroad in charge of construction on the Sea Isle City Branch until the autumn of 1883, when he was put in charge of the piers, wharves, and freight-house construction at Canton, Baltimore, Md., until February, 1884. He was then tranferred to the Maintenance-of-Way Department, and subsequently was attached, for one year, to the General Superintendent's office as Assistant Engineer in charge of surveys in the field. In 1886, he was transferred to Pittsburg, Pa., in charge of the reconstruction of the Pittsburg Yard, in connection with the introduction of interlocking switches and signals.

In February, 1888, Mr. Covode left the service of the Pennsylvania Railroad and became Chief of Party on the location of the projected Nicaragua Canal, under General Menocal, on which work he remained for nearly two years. In 1889, he was invited to become Consulting Engineer for Don Eduardo Casey, a wealthy citizen of the Argentine, who was engaged in large undertakings in that Republic. Mr. Covode retained this position until the failure of Baring Brothers put a temporary end to development plans in the Argentine Republic.

Returning to the United States, Mr. Covode became interested in contracts for building a portion of the Midvale Branch of the Pennsylvania Railroad. In 1893, he associated himself with the California Alcatraz Asphalt interests, and introduced this pavement into a number of Eastern cities. He continued to be interested in the Asphalt Company until the formation of the Trust in 1899.

After 1899 Mr. Covode became engaged in mining operations in California, Nevada, and Mexico, in which he continued until his death, which was caused by over-exertion in high altitudes in Bolivia, South America. He had gone to Bolivia, in the interest of a New York syndicate, to look over the mining developments in the Andes, travel-

* Memoir prepared by A. H. Renshaw, M. Am. Soc. C. E.

ing into altitudes of 20 000 ft., and on his return to La Paz, he died suddenly from the effect of the undue strain on his heart.

In February, 1897, he married Miss Louise Bournonville Moulder, of San Francisco, Cal., who alone survives him.

Mr. Covode was a worthy son of a worthy father, and deserved the honorable title of "Honest" in the same sense that his father had earned it in war times. He also had a very lovable character. His self-sacrificing loyalty to friends and to those who trusted themselves or their interests to him, was the most marked of his many sterling qualities, leading him too frequently to subordinate his interests and welfare to those of others.

He was an able Engineer, and his varied experience would have fitted him for many years of usefulness in the Profession, but for his untimely death.

Mr. Covode was elected a Member of the American Society of Civil Engineers on April 2d, 1890. He was also a member of the San Francisco Association of Members of the American Society of Civil Engineers, of the Engineers and Delta Phi Clubs of New York City, and of the English Club of Monte Video, Argentine Republic.

HENRY HARDING, M. Am. Soc. C. E.*

Died October 23d, 1910.

Henry Harding was born on December 10th, 1837, at Hartland, Vt., where his father, Dr. John Harding, was a physician for many years.

He studied engineering under Mr. Job Atkins, a mining engineer of Richmond, Va., during 1859-60, and was afterward engaged on the construction of the Hudson River Railroad.

From 1865 to 1870, Mr. Harding was employed, under General Grenville M. Dodge, on the survey and construction of the Union Pacific Railway. He was also employed on the Adirondack, Housatonic, and Naugatuck Railroads, in charge of construction and in various other capacities.

From 1871 to 1895, he was engaged at intervals by the United States Corps of Engineers in charge of river and harbor improvements and other work, and was wont to recall with especial pride the construction of the fortifications of Fort Adams, at Newport, R. I., in 1871-73.

While in the employ of the Government, Mr. Harding contracted malaria, and, in 1895, he retired to his home at Hartland Four Corners, Vt., where he continued to live until his death.

Mr. Harding was a man of wide acquaintance and high reputation in his Profession, to which he was devoted, and, although living

*Memoir prepared by the Secretary from information on file at the House of the Society.

in a secluded country village, he kept well abreast of all the improved methods and was familiar with all new instruments used in engineering work. At the time of his death, he was engaged as Engineer in charge of the construction of the new sewerage system of Windsor, Vt.

Mr. Harding was the embodiment of painstaking accuracy and scrupulous honor, and any work done under his superintendence was honestly constructed and fully served its purpose. He was of a genial and courteous manner, modest, kind-hearted, and drily humorous, an agreeable and interesting social companion. He never married, and is survived by several nephews and nieces.

Mr. Harding was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

EVELYN PIERREPONT ROBERTS, M. Am. Soc. C. E.*

DIED DECEMBER 30TH, 1910.

Evelyn Pierrepont Roberts was born at Fort Leavenworth on Christmas Day, 1848, when what is now the State of Kansas was the Indian Territory and Fort Leavenworth one of the extreme outposts of the civilized frontier of the United States. His father was General Benjamin Stone Roberts, U. S. A., and his mother, Elizabeth Pierrepont Sperry. On both sides he was descended from distinguished ancestry. The first Roberts came from England about 1620, and his descendants took part in the Revolution and various Indian Wars. On his mother's side the line runs back to Evelyn Pierrepont, Duke of Kingston, whose son, James Pierrepont, was the first clergyman to settle in New Haven, Conn., and became one of the founders of Yale College.

In 1870 Mr. Roberts was graduated from the Sheffield Scientific School of Yale, and went West. He found employment on the staffs of the Northern Pacific Railroad west of the Rocky Mountains, the Canadian Pacific Railway in British Columbia, and the Spring Valley Water-Works, of San Francisco, Cal.

Returning East he was employed on the Croton Aqueduct, and subsequently acquiring some valuable granite properties, he took up masonry contracting as well as maintaining practice as a Consulting Engineer. He constructed the foundation for the elevated portion of the New York Subway and the great granite arches of the Cathedral of St. John the Divine, in New York City, which latter he always considered his greatest achievement.

Notwithstanding his many business interests, Mr. Roberts found time for altruistic work, especially the obtaining of employment for those unfortunates who had served prison sentences and who appeared to be anxious to begin life anew and earn an honest livelihood. Of

* Memoir prepared by William Barclay Parsons, M. Am. Soc. C. E.

this work he spoke only to his intimates, and, consequently, the great good and charity he did was known to few, in many cases only to the recipients.

Mr. Roberts was married July 5th, 1903, to Helen Frances Caleb, who, with one son, survives him.

He was a member of the Century, Yale, and Church Clubs, the National Geographic Society, the Military Order of the Loyal Legion, Sons of the Revolution, Fraternity of Delta Psi, and many other similar organizations. He was also a Director of the Danville and Mt. Morris Railroad; President and General Manager of the Mohegan Granite Company; and Director of the Jerry McAuley Cremorne Mission.

Mr. Roberts was elected a Member of the American Society of Civil Engineers on May 7th, 1884.

JOHN WRIGHT SEAVER, M. Am. Soc. C. E.*

Died January 14th, 1911.

John Wright Seaver, was born in Madison, Wis., on January 8th, 1855. He was descended from pure New England Revolutionary stock, the first Seaver coming to America shortly after the *Mayflower* Pilgrims. While very young, his parents moved to Buffalo, N. Y., where his father died, and where he received a public school education. At the age of thirteen, he left school to learn the machinist's trade. He entered the machine shop of the Shepard Iron Works, where he spent two years, and was then transferred to the drafting-room. His technical education was obtained at this time under the greatest difficulties by attending night school, which was three miles from his home, and to which he walked after working hard all day. He read such trade journals and engineering books as he could secure, and one of his treasured possessions was a copy of Haswell's "Engineers' and Mechanics' Pocket-Book," given him by his mother when he was fourteen.

At the age of eighteen, after being with the Shepard Iron Works for four years, Mr. Seaver entered the machine shop of the Howard Iron Works, and at twenty he was employed by the Buffalo Car Company as Assistant Superintendent in charge of about 1 000 men. He left this Company to become a member of the firm of Seaver and Kellogg, and at this time designed and built the first steel cars in the United States. These cars were not commercially successful, as they were too far ahead of their time.

After spending one and one-half years in this business, he became Assistant Engineer of the Kellogg Bridge Works. He made structural iron a specialty, and began to attract outside attention as an Engineer.

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*Memoir prepared by J. E. A. Moore, M. Am. Soc. C. E.

In 1880 Mr. Seaver moved to Pittsburg, Pa., and became Chief Engineer of the Iron City Bridge Works, for which firm he designed and built a number of notable bridges and other steel structures. In 1884 he became the Chief Engineer for the Riter-Conley Manufacturing Company, and in this capacity his reputation became world-wide. He designed and built blast furnaces, steel works, oil refineries, gasometers, buildings, bridges, and other steel constructions. He was instrumental in building up this concern to be one of the greatest of its kind in the world.

In 1896 Mr. Seaver, with S. T. and C. H. Wellman, formed the Wellman-Seaver Engineering Company, which finally became the Wellman-Seaver-Morgan Company. While with this Company he was Vice-President and Chairman of the Board, and a Director until his death.

After being with this Company for ten years, Mr. Seaver became associated with J. E. A. Moore, M. Am. Soc. C. E., as Consulting and Contracting Engineers, with offices in Cleveland, Ohio. He continued in this work until his death.

Mr. Seaver's engineering experience was most varied, including the civil, mechanical, marine and mining branches of the profession. While in Buffalo he designed and built many large marine engines, the principal of which was that for the steamer *Great Western*, which was a remarkable vessel at that time. He designed and built the first steel head frame, and the first gantry crane used in America, and also, as previously stated, the first steel cars. He was an authority on structural steel construction of all kinds, as well as material handling, steel and iron manufacturing, and coke-oven machinery, and compiled the first standard steel railroad bridge specification, which was adopted by some of the large railroad companies.

The principal traits in Mr. Seaver's character were his resourcefulness and perseverance, and it can be said of him that he never gave up. An instance of his resourcefulness was displayed in Buffalo when it was discovered that, after the hull of a large vessel was ready for launching, the water was too low in the slip to float it; in order to overcome this, he tied several tugs across the mouth of the slip, and by working their propellers, the water was raised sufficiently to float the vessel, and she was launched successfully. His genial, sunny disposition made for him many close, warm friends, both in this country and abroad, and he was beloved by all who knew him. His liberality was his worst fault.

Mr Seaver died suddenly of apoplexy at his residence in Cleveland, Ohio, on January 14th, 1911, and is survived by his wife and four children, an aged mother, and two sisters.

Mr. Seaver was elected a Member of the American Society of Civil Engineers on November 6th, 1901. He was also a Member of the American Society of Mechanical Engineers, the Cleveland Engineering Society, and many other business and social organizations.

EDWIN MERRITT HOLMES, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 11TH, 1911.

Edwin Merritt Holmes was born at Colorado Springs, Colo., on June 20th, 1878. He received his education at the 104th Street Public School, New York City; the Mattoon High School, at Mattoon, Ill.; the College of the City of New York, and Cornell University.

From 1898 to 1901, Mr. Holmes was employed, in the United States, on timber boundary surveys and in the construction of tram and narrow-gauge logging roads. He also had charge of the landscape gardening for a summer hotel, including an artificial lake, pavilions, etc., as well as the reconstruction of a dam for a gravity water supply.

From June, 1901, to May, 1902, he was with the Engineer Department of the Military Government of Cuba, at Santiago, as Rodman, Leveler, and Transitman on city improvements. He was also employed as Street Inspector in charge of repairs to all city property and the harbor dredge. During May and June, 1902, he was connected with the Department of Public Works of the Cuban Government, as Assistant Engineer and Inspector in charge of the construction of macadam roads and a concrete bridge.

Although a Civil Engineer by profession, Mr. Holmes turned his attention to mining, and in June, 1902, entered the employ of the Spanish-American Iron Company, at Daiquiri, Cuba, as Assistant Engineer in charge of designing and constructing appliances for handling the product of the mines, of opening new deposits of iron ore for the extraction of sample cargoes, etc. In August, 1906, he was made Assistant General Superintendent of the Company, in direct charge of all engineering work, together with other duties connected with the operation of the mines.

On February 1st, 1909, Mr. Holmes was appointed General Superintendent of the Juragua Iron Company, and took charge of the operation of its mines at Firmeza, Cuba, which position he filled with conspicuous ability until his death.

Mr. Holmes was not married. He is survived by his father, Rev. Benjamin F. Holmes, of Clairmont, Va., two brothers, Mr. Robert L. Holmes, of San Juan, Porto Rico, and Mr. F. C. Holmes, of Firmeza, Cuba, and one sister, Mrs. Paul English, of Muskogee, Okla.

Mr. Holmes was elected a Junior of the American Society of Civil Engineers on March 31st, 1903, and an Associate Member on January 2d, 1907. He was also a Member of the American Institute of Mining Engineers and of the American Society of Engineering Contractors.

*Memoir prepared by DeB. Whitaker, Esq., Vice-President and General Manager, Juragua Iron Company, Santiago de Cuba, Cuba.

Vol. XXXVII. APRIL, 1911. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXVII. APRIL, 1911. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TWO EARTH DAMS OF THE UNITED STATES RECLAMATION SERVICE.

By D. C. Henny, M. Am. Soc. C. E.

To be Presented June 7th, 1911.

The object of this paper is to describe in some detail two earth dams built by the United States Reclamation Service, and to compare the different methods used in their construction. The two dams are the Cold Springs Dam, in Oregon, and the Conconully Dam, in Washington.

Cold Springs Dam.

The Cold Springs Dam is part of the works of the Umatilla Project, which comprises a tract of land lying south of the Columbia River and east of the Umatilla River, encompassing the Town of Hermiston, 192 miles due east of the City of Portland.

The principal dimensions of the dam are:

Greatest height above bottom of creek channel	98.5	ft.
Height above valley bottom	87.5	"
Width of valley on center line of dam	400.0	"
Length of crest of dam	3 800.0	"
Top width	20.0	"
Length of spillway	330.0	"

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Up-stream slope........................	3:1
Down-stream slope......................	2:1
Total volume of dam....................	673 200 cu. yd.
Capacity of inlet canal, at reservoir........	300 sec-ft.
Capacity of outlet canal..................	225 sec-ft.
Capacity of reservoir above outlet gates.....	49 000 acre-ft.
Area of reservoir at high-water level........	1 590 acres.
Drainage area above dam..................	210 sq. miles.

Source of Irrigation Supply.—The Columbia River, bordering on the irrigable lands to the north, has its water surface from 140 to 240 ft. below the general ground surface. Its grade is very flat, and gravity diversion from this source, therefore, is entirely out of the question. The Umatilla River has a fairly steep grade, averaging 15 ft. per mile in its lower 25-mile course, and presents the only possibility for gravity diversion.

From November 1st to July 1st the flow usually varies from 50 to 5 000 sec-ft., March and April being the flood months. During the summer the flow is less than 50 sec-ft., and is fully appropriated by prior irrigators.

It was computed that with the water which might be directly diverted for irrigation during the flood season, a storage supply of 49 000 acre-ft. would be sufficient for the irrigation of the 25 000 acres of irrigable land embraced in the project, considering also a prior water right which had been purchased.

Reservoir Site.—On the main river, above the point of desired diversion, a reservoir site could not be found, but a suitable one was located in the small adjoining basin of Cold Springs Creek, above and close to the lands to be irrigated, as shown on Fig. 1. This basin has an area of 210 sq. miles, and presents a gently rolling topography.

The precipitation on the water-shed of the creek ranges from 6 in. for the lower, to 18 in. for the higher, lands, much of which is dry-farmed for grain. The land has a generally deep and retentive soil, the rock crops out at only a few points, and the slopes are so flat, especially near the divides, that a survey was required to establish the limits and area of the water-shed with some degree of certainty.

The creek is usually dry throughout the year, and all evidence indicated a very limited flow at any time. There was some underflow,

however, and small springs break out lower down, from which the creek probably derived its name. Usual absence of surface run-off is explained by the small rainfall, the gently sloping character of the

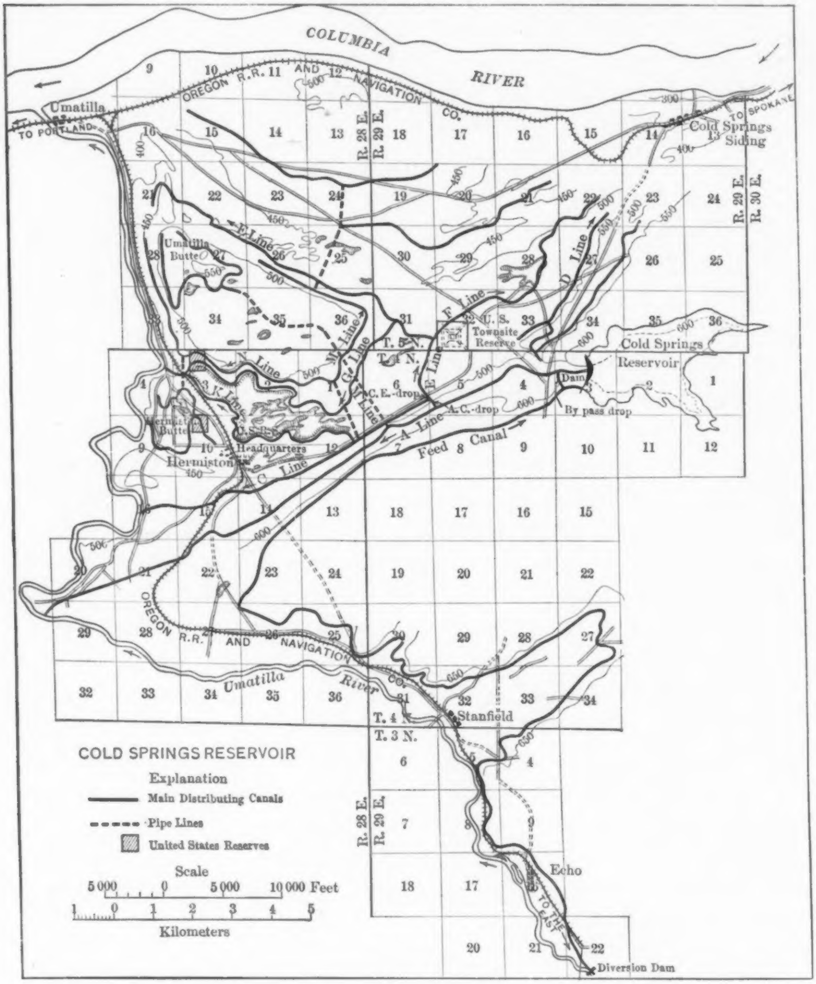


FIG. 1.

ground surface, and the absorbent and retentive character of the soil. The reservoir is supplied by a feed canal from the Umatilla River, with a capacity of 300 sec-ft. at the reservoir and a length of 25 miles, presenting no special difficulties. It heads about 1½ miles above the

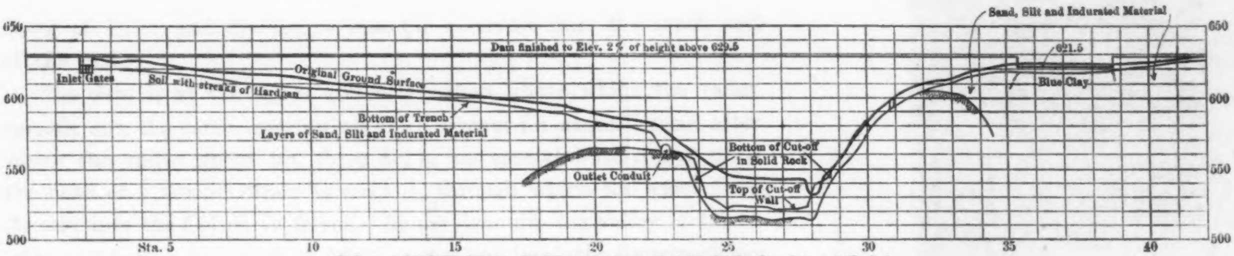


FIG. 1. LONGITUDINAL SECTION OF CUT-OFF TRENCH ON LINE A-A

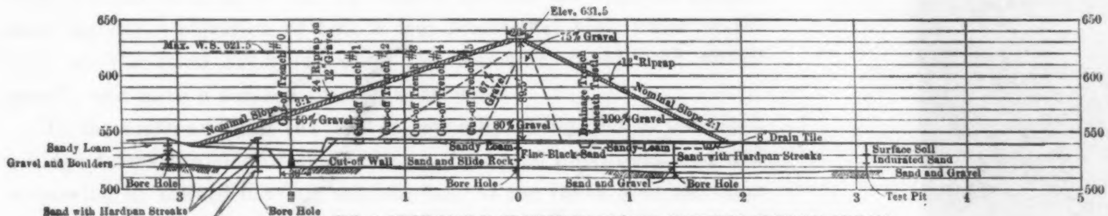


FIG. 2. SECTION B-B AT STATION 27+93.8–MAXIMUM SECTION OF DAM

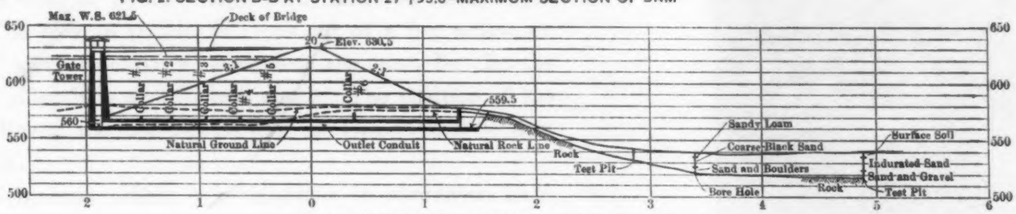


FIG. 3. SECTION C-C THROUGH OUTLET CONDUIT

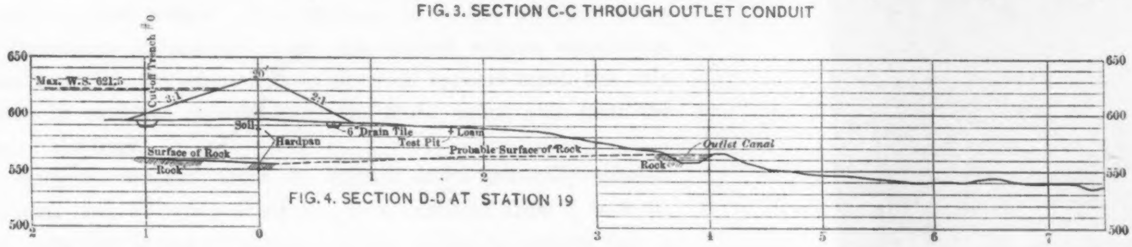


FIG. 4. SECTION D-D AT STATION 19

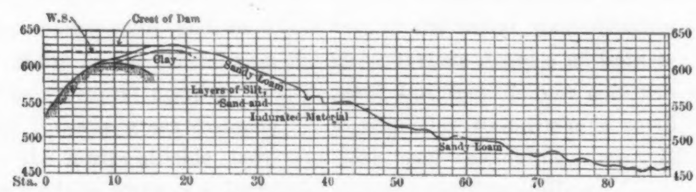


FIG. 5. SECTION THROUGH RIDGE AT NORTH END OF DAM ALONG LINE E-E

SECTIONS OF
COLD SPRINGS DAM.

Town of Echo, and discharges into the reservoir near the south end of the dam.

The dam is at the upper end of a cañon, above which the valley spreads out, the cañon continuing down stream for about 1 mile, after which the valley opens up, there being no special watercourse until the head of a second cañon is reached, through which the water finds its way into the Columbia River. At the dam site the valley is 400 ft. wide and has an elevation of 542 ft. above sea level, the bottom of the creek channel being at 531 ft. In order to produce the required storage above an outlet elevation of 560, it was found that high water would come at Elevation 621.5 ft., when the reservoir would hold 49 000 acre-ft. and have a surface area of 1 590 acres.

To the south and east the land is generally rising, but to the north a ridge forms the side of the reservoir, broad at its eastern end and narrowing up toward the dam site, the base at the valley floor elevation being about 1 mile wide at its narrowest point and 450 ft. at high-water elevation. The rock in this ridge crops out at a few points on the reservoir side, and evidently rises toward the east, but in the narrow part of the ridge it is at unknown depth. The position of the rock on the center line of the dam is shown in the longitudinal section, Fig. 1, Plate XXIX.

The rock is of igneous origin, non-columnar in character, hard, sound, and of good weight. On blasting, it breaks up into sharp angular fragments of assorted sizes, the largest seldom containing more than 1 cu. ft. On exposed faces the rock appears tight, the only open seams in evidence occurring on the south cañon side near the lower toe of the dam.

The overlying material, from the surface down, generally consists of fine, loamy soil, of sandy character, to a depth of from 2 to 8 ft., fine sand, irregular layers of silt from a few inches to several feet in thickness, coarser sand, and, in the bottom of the cañon, water-bearing gravel. Many of the strata were found in various degrees of induration. Their general position is horizontal, but they do not continue unbroken for any great distance. The only clay found was a small body on the north hillside, hard and blue, and resembling slate. It has been made to serve as a foundation for the overflow spillway.

The longitudinal profile of the dam shows that while it is feasible to make tight juncture with the rock crossing the cañon and on the

cañon sides, this is not the case at some distance away, where the rock drops beyond economical reach.

From the foregoing it may be noted that the site is not a very desirable one, from the standpoint of assured water-tightness. Assuming a perfectly water-tight dam to be built, connecting with bed-rock across the cañon and reaching to depths of from 15 to 6 ft. on the side-hill, it is apparent that leakage from the reservoir remains possible through the seams in the rock, through the layers of sand on each side of the cañon below the cut-off line, as shown by the profile, Fig. 4, Plate XXIX, and through the ridge to the north of the reservoir in the vicinity of the dam, as shown by the profile, Fig. 5, Plate XXIX. These possibilities of leakage were carefully studied.

As regards the rock, volcanic formations in the Northwest are generally very changeable within short distances, and the absence of seams and the soundness of the rock within the range of local borings give no assurance of its character within a short distance beyond. This condition prevails throughout large areas in Oregon, Washington, and Idaho, and forms one of the elements of risk in connection with all reservoir construction in that region.

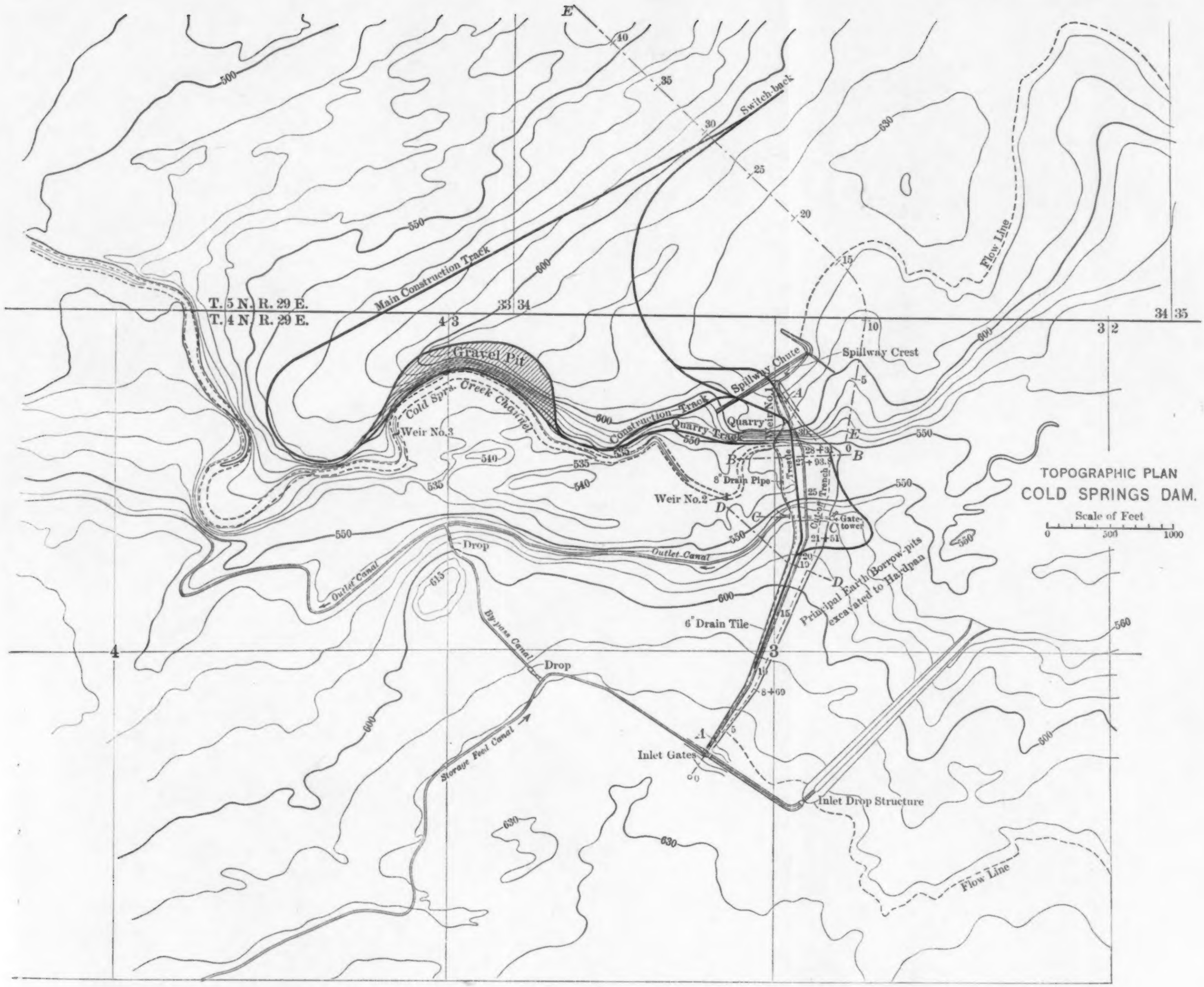
As regards the overlying material, movement of water through some fine sand layers was to be expected, both under the dam and through the north ridge. In either case it was argued that the path to be followed by the water would be circuitous and long, as compared with the head. Thus velocities would be held down to ordinary percolating limits, such as would be utterly incapable of carrying the granular material of which the soil is mostly composed.

As regards the economical value of possible leakage water, the reservoir will be filled slowly during the winter, but the bulk of the river water is not available for reservoir supply until near the beginning of the irrigation season, when leakage under and around the ends of the dam, which would gather in the cañon below, can be diverted and used for irrigation.

Available Material for Dam Construction.—There were available for dam construction, within reasonable distance, the following classes of material:

Basaltic rock, hard and sound, readily blasted, and quite suitable for rip-rap;

PLATE XXX.
PAPERS, AM. SOC. C. E.
APRIL, 1911.
HENNY ON
TWO EARTH DAMS
OF THE U. S. RECLAMATION SERVICE.



Gravel in deep hillside deposits on the north side of the cañon, ½ mile below the dam, as shown on Plate XXX;

Fine sandy loam, most readily available in any direction, for use in the embankment;

Pure volcanic ash, in occasional strata and in small quantities;

Indurated clay deposits, principally at the north end of the dam, and limited in area;

Sand and gravel, in deeper strata underlying the surface soil, and to some extent indurated.

The first three were the only ones which could be obtained in large quantities without removing large masses of overlying materials. In order to judge of their suitableness for dam construction, a series of percolation experiments was made. These experiments have been fully described in an article* by E. G. Hopson, M. Am. Soc. C. E., and the writer.

Five classes of material were tested in an 18-in. tank similar to that used in experiments by the Massachusetts State Board of Health. In addition, tests were made with various mixtures of fine subsoil and gravel. Physical characteristics, specific gravity, and coefficients of fineness and uniformity, were also determined. The results, from the article mentioned above, are quoted in Table 1, which has been corrected. These results are also shown on Fig. 2.

TABLE 1.—PERCOLATION EXPERIMENTS, COLD SPRINGS DAM.

Sample.		Specific Gravity.		Percentage of Voids.			Uniformity coefficient.	Effective size, in millimeters.	Rate of percolation, in gallons per acre per day.
				Dry.		Wet.			
		Constituents.	Mass.	Loose.	Compact.	Rammed.			
(A) Surface soil....		2.52	1.41	59	49	44	3.9	0.023	180 000
(B) Fine subsoil...		2.79	1.55	54	43	45	4.0	0.025	350 000
(C) Pit gravel......		2.90	1.91	42	37	34	9.2	0.559	155 000 000
(D) Volcanic ash...		2.66	0.94	74	68	65	5.5	0.016	400 000
(E) Coarse subsoil.		2.93	1.57	55	49	46	1.5	0.079	650 000
Mixtures:									
B	*C*								
75%	25%	2.64	1.75	50	39	38	3.0	0.066	60 000
67%	33%	2.83	1.76	47	41	39	....	.....	30 000
50%	50%	2.90	1.91	42	33	34	14.7	0.076	20 000
33%	67%	2.87	1.95	41	35	32	20.0	0.076	30 000
25%	75%	2.83	2.01	47	43	29	21.0	0.086	30 000
20%	80%	2.91	2.04	47	40	30	....	0.091	70 000
15%	85%	2.84	1.88	41	35	34	18.0	0.102	1 000 000

* *Engineering News*, March 7th, 1907, p. 250.

In explanation, it should be stated that the specific gravity of the mass refers to the material in its most compact form in dry condition. For the determination of the percentage of voids in the dry loose material, it was poured into the tank from a scoop when perfectly dry, and for the dry compact material, it was rammed into thin layers with a stick having a 2 by 2-in. base. The wet rammed material was similarly deposited after such degree of moistening as was believed to result in the tightest mass. The uniformity coefficient and effective size were determined by the Massachusetts State Board of Health standard.

The pit gravel is made up principally of coarse sand, containing about 10% of pebbles and cobbles. It has a far smaller percentage of voids than the finer material, due to a more favorable graduation of sizes. It also shows the highest specific gravity, both as to its ingredients and as to the mass, thus fitting it well for use in a dam, to furnish stability and perfect drainage.

The rate of percolation is for a head of 3 ft. through 3 ft. of material on the third day after the commencement of the test, and is corrected for temperature on the basis of the Hazen formula.

It will be noted from the results of the experiments that the addition of subsoil to the gravel has a very marked effect in reducing the rate of percolation. Practical water-tightness was obtained by a wide range of proportions, there being little difference in the rate of percolation between a mixture containing 25% and one containing 66% of soil. Thus the exact proportion of the mixture within these ranges was not essential, which is an important consideration in the practical work of dam construction. The greatest tightness was found with a proportion of 50% gravel and 50% soil, with which the rate of percolation, as compared with that of soil alone, was found to be less than 6%, and as compared with gravel alone less than 0.03 of 1 per cent. After careful study of the results of these experiments, a section of dam was adopted, as shown on Fig. 2, Plate XXIX. Stability was secured by the use of gravel throughout the entire section of the dam; water-tightness by an admixture of fine subsoil in the up-stream portion of the section; and perfect drainage by the use of unmixed gravel in the down-stream portion.

The cut-off trench across the bottom of the cañon is 29 ft. deep and 30 ft. wide at its connection with bed-rock. It reduces in depth

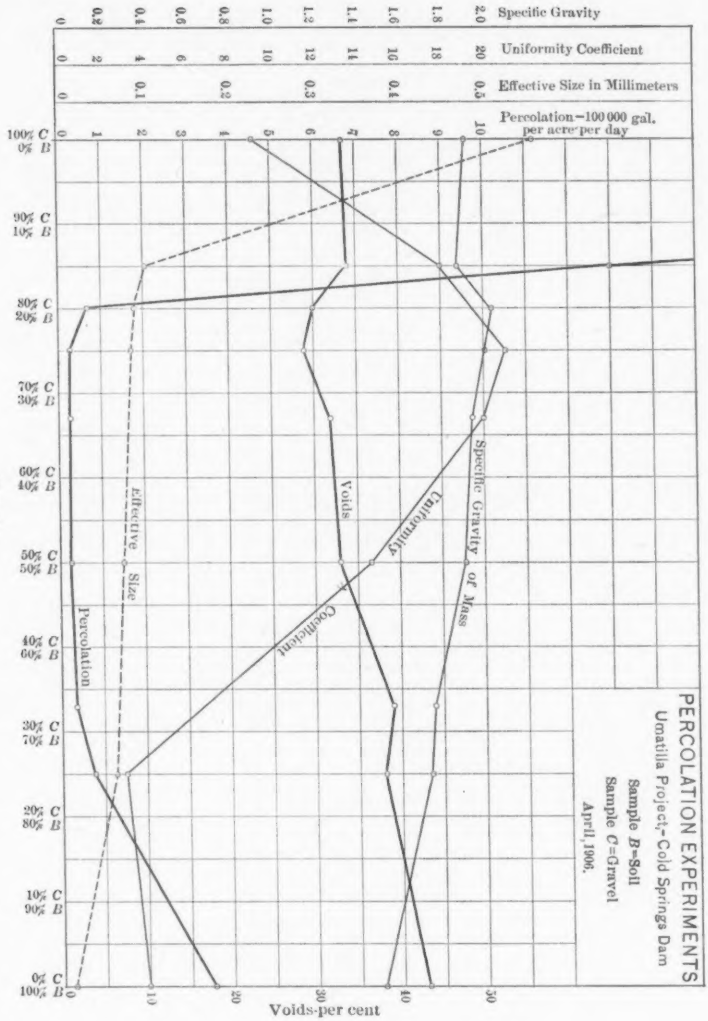

PERCOLATION EXPERIMENTS
Umatilla Project,-Cold Springs Dam
Sample B=Soil
Sample C=Gravel
April, 1906.
Specific Gravity
Uniformity Coefficient
Effective Size in Millimeters
Percolation-100 000 gal. per acre per day
Voids-per cent
Voids
Specific Gravity of Mass
Uniformity Coefficient
Effective Size
Percolation
100% C 0% B
90% C 10% B
80% C 20% B
70% C 30% B
60% C 40% B
50% C 50% B
40% C 60% B
30% C 70% B
20% C 80% B
10% C 90% B
0% C 100% B

FIG. 2.

and width up the hillsides until it is 6 ft. deep and has a bottom width of 10 ft. at the ends of the dam. To retard the flow along the plane of contact with bed-rock, a thin cut-off wall, 7 ft. high, was constructed on the center line of the trench, across the cañon, reducing in height up the side-hill. Five additional walls were built on the north hillside where the rock was exposed.

Provision was made for drainage by a gravel-filled trench along the entire down-stream toe of the dam with tile drain, and a network of additional trenches under the high portion of the dam, consisting of a parallel trench 120 ft. up stream from the water toe, with cross-trenches every 100 ft.

Slope Protection.—The water face of the dam is covered with 12 in. of pit gravel and 24 in. of rock rip-rap. The down-stream face is covered with 12 in. of rock rip-rap principally to prevent the gravel from being displaced and as a protection against wind action.

Inlet Structure.—The feed canal enters the reservoir near the south end of the dam. No rock could be found over which to drop the water, and a protective structure therefore became necessary. This consists of two continuous reinforced side-walls, with bottoms stepped and tops sloping, with cross-walls forming water-pockets, floored with concrete and covered with wooden wearing floors, the water spilling over each cross-wall and dropping into the water-cushion formed by the next cross-wall. There are nine water-steps, with a vertical drop of 6 ft. each, the lowest cross-wall having its top at low-water elevation. At the top of the structure the side-walls spread apart to accommodate a steel-plate weir with a 24-ft. crest.

This drop structure was built in deep excavation, it being left to the incoming water to overflow through a training furrow and cut its own channel to low water in the reservoir. This was partly accomplished in the course of three weeks by a flow varying from 30 to 100 sec-ft. Several steps still remain buried, owing to the grade assumed by the bottom of the cut. The sides of this cut stood at first vertical, but have gradually slumped, and are tending to a uniform slope.

About ¼ mile up stream from this structure, following the canal line, and where the canal alignment crosses the extended center line of the dam near its south end, a concrete gate structure has been built, with four arched openings, 6 ft. 6 in. wide in the clear, with hinged floating gates to prevent the reservoir water, when raised above the

spillway height by cloudbursts, from running back into the canal and overtopping its banks.

Flash-boards are also provided for backing the canal water over a concrete crest in the canal bank some distance up stream into a lined channel, affording direct connection between the inlet and outlet canals, and the means of by-passing the reservoir.

Outlet Conduit and Gate-Tower.—The outlet conduit and gate-tower are located on bed-rock on the south side of the cañon, with outflow elevation at 560 ft., or approximately 17 ft. above the cañon bottom. There are two 4 by 4-ft. Coffin sluice-gates, placed in series, one at the outside of the gate-tower and the other at the inside, the former being an emergency gate.

The conduit is founded on rock, and there are cut-off collars at intervals of 30 ft., the conduit discharging into a canal cut in the rock for about 200 ft., below which it continues in earth. An overflow spillway is provided in the canal bank, cut in the rock section, the spill crest having concrete guide-walls to the bottom of the cañon. The outlet canal capacity is 225 sec-ft. The gate-tower at the upper end of the conduit stands near the water toe of the dam. It is built of concrete, slightly tapering, and is connected with the top of the dam by a steel footbridge.

Spillway.—On the hard clay bed on the north hill shoulder, a spillway was built, with a crest length of 330 ft. and a crest elevation of 8 ft. below the top of the dam. At each end there are abutment walls, with cut-off walls reaching into the body of the dam. The water, after spilling over the crest, slides down an inclined concrete-paved slope and enters a concrete-lined channel with a general direction parallel to the overflow crest and at right angles to the dam. This channel is excavated in thorough cut past the line of the dam, through hard clay and fine sand formation, and terminates in hard rock at an elevation of 40 ft. above the cañon bed, the end being provided with a heavy cut-off wall extending well into the rocky sides.

The entire spillway channel has a concrete lining built in disconnected sections to 10-ft. squares, with joints underlaid by concrete cut-off walls. Where it passes through sand formation, it is reinforced with $\frac{3}{8}$-in. bars laid in 12-in. squares, and has weep-holes on the dam side. The capacity of the spillway, with 3 ft. of water over the crest, is assumed at 5 900 ft., at which point the dam will have a freeboard

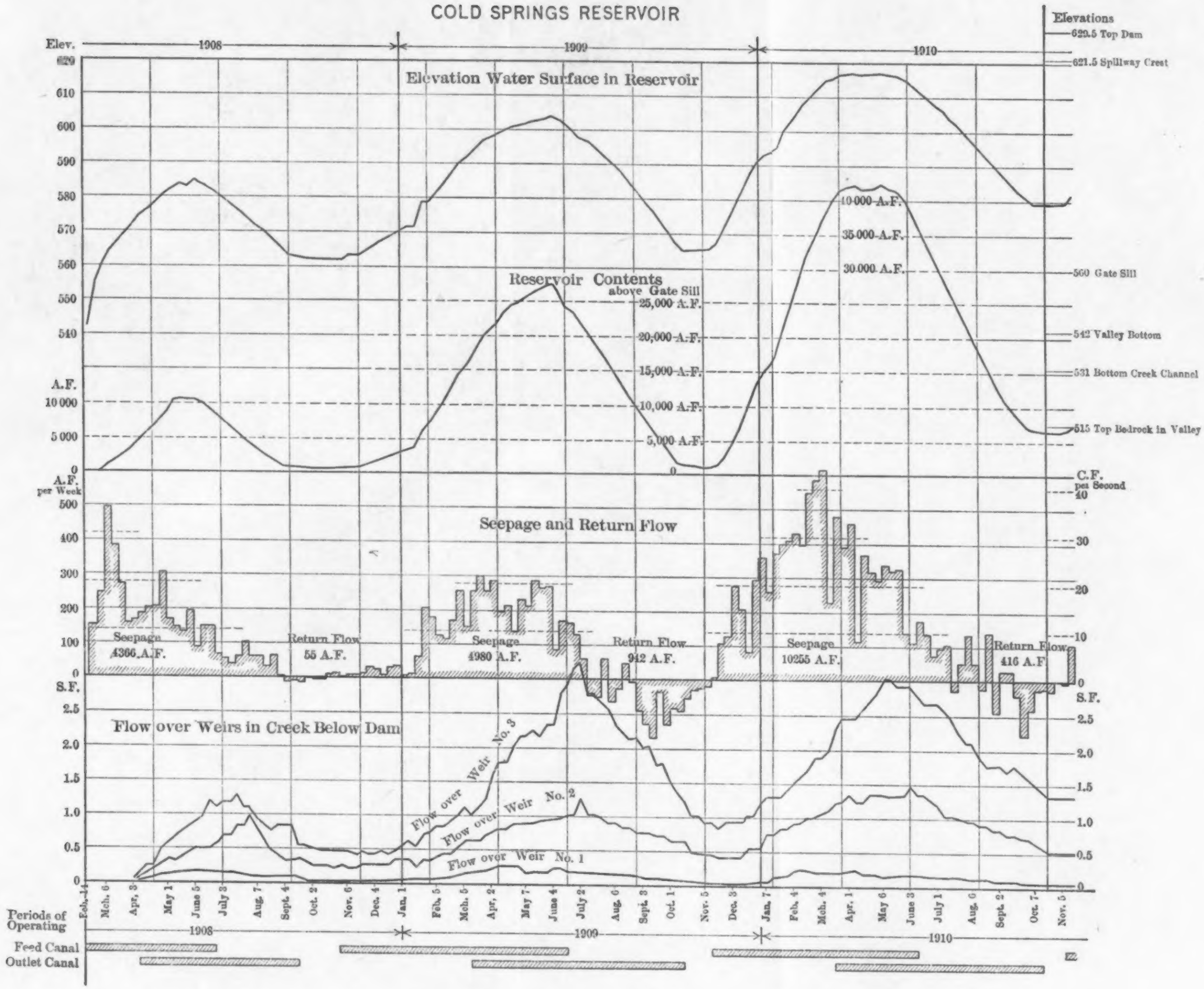
of 5 ft. independent of a super-elevation of 2% of the height for possible settlement.

There had been no observed run-off from the drainage area of the reservoir prior to the commencement of the construction of the dam. During January, 1907, when the construction trestle was being erected, a heavy rain occurred, falling on 10 in. of snow, with the ground beneath frozen by previous cold weather. Three floods followed in rapid succession, discharging in the aggregate 15 000 acre-ft., and peaking for 3 hours at 6 000 sec-ft. Approximately 10 000 cu. yd. of material were eroded within the lines of the dam base. Floods of this type can only occur in the winter, at which time the reservoir can be but partly filled. In summer, cloudbursts may be expected, but their aggregate run-off is usually small—the disastrous Heppner cloudburst of 1903, for instance, resulting in an aggregate flow estimated at only 1 600 acre-ft.

With a spillway capacity of 5 900 sec-ft. at 3 ft. depth on the crest, the reservoir has a capacity, above the crest, of 7 000 acre-ft. The floods of 1907, above mentioned, were entirely unlooked for, but fortunately did no damage other than eroding part of the dam foundation. The creek channel would not ordinarily accommodate in excess of a few hundred second-feet, and the heavy growth of sage-brush on its banks indicated absence of any recent large flow. This is one of the many illustrations of the danger involved in estimating maximum floods from drift lines or other surface appearances.

Method of Construction.—The design called for the handling of approximately 490 000 cu. yd. of gravel from the gravel pit in the north cañon side, about ½ mile below the dam; for the excavation of 191 000 cu. yd. of loam or subsoil, to be obtained mostly from the slopes within the reservoir; and for about 36 000 cu. yd. of rock pitching, to be placed on both slopes.

The gravel was excavated with a 70-ton, Model No. 60, Marion steam shovel, with 2½-cu. yd. bucket. The rolling stock consisted of fifty 4-yd. side-dump cars, and five 16-ton American locomotives running on a 3-ft. gauge track of 35-lb. rails. The track system is shown on Plate XXX, the average distance of gravel haul being 2¼ miles, and the maximum grade being 1¼ per cent. The gravel in the pit rose to from 30 to 60 ft. above the shovel, and was in places overlaid with considerable soil, rendering it necessary to watch the proportions of soil and gravel as they came on the cars to the dump.


COLD SPRINGS RESERVOIR
Elev.
1908
1909
1910
Elevations
629.5 Top Dam
621.5 Spillway Crest
560 Gate Sill
542 Valley Bottom
531 Bottom Creek Channel
515 Top Bedrock in Valley
Elevation Water Surface in Reservoir
Reservoir Contents
above Gate Sill
40,000 A.F.
35,000 A.F.
30,000 A.F.
25,000 A.F.
20,000 A.F.
15,000 A.F.
10,000 A.F.
5,000 A.F.
A.F.
A.F. per Week
C.F. per Second
Seepage and Return Flow
Seepage 4366 A.F.
Return Flow 55 A.F.
Seepage 4980 A.F.
Return Flow 942 A.F.
Seepage 10255 A.F.
Return Flow 416 A.F.
S.F.
Flow over Weirs in Creek Below Dam
Flow over Weir No. 3
Flow over Weir No. 2
Flow over Weir No. 1
Periods of Operating
Feed Canal
Outlet Canal

The shovel was served by four gravel trains, of from nine to twelve cars each, which handled on an average, including moving and delays, about 1 600 cu. yd. per shift of 8 hours, the output per shift occasionally exceeding 2 200 cu. yd. The total rise from the steam shovel to the top of the dumping trestle was 65 ft., and the coal consumption per locomotive, for 1 149 shifts of 8 hours, averaged 2 400 lb., coal being obtained from the Kemmerer mines in Wyoming.

The gravel was delivered on the dam by dumping from a trestle, 65 ft. high, built across the cañon, with its center line about 60 ft. down stream from and nearly parallel to the center line of the dam. The entire trestle came within the 100% of the gravel section, and the posts were left buried in the gravel, but all bracing was removed as the work progressed.

The fine subsoil or loam was obtained from various sources, as follows:

From surface layers overlying the gravel in the gravel pit..................	100 000 cu. yd.
From borrow-pits on the side-hill, up stream from the dam..............	108 600 " "
From the feed canal and trenches........	40 700 " "
From the spillway channel.............	17 700 " "
Total.........................	267 000 cu. yd.

The loam from the gravel pit was handled in the same manner as the gravel. The loam from the borrow-pits was handled by wheel-scrapers up to distances of 500 ft. (22 300 cu. yd.), and by dump-wagons loaded by an orange-peel excavator for greater distances up to 2 000 ft. (86 300 cu. yd.). The loam from other sources was moved by scrapers, only that portion excavated from trenches, etc., being used which was found suitable, the remainder being wasted.

The loam was delivered first, in its proper proportions, and spread by a road scraper, after which gravel was spread over it, being scraped by Fresno scrapers from the foot of the gravel dump near the trestle. The materials were mixed at first with disk-harrows and subsequently with cultivators, the points of which scraped over and into the top of the previously mixed and rolled layer, after which the material was watered and rolled, producing a final layer of from 4 to 5 in. compacted into a hard mass which required picking to excavate. Constant

watch was kept of the thoroughness of the mixing process, by excavating test-pits. It was found impossible to secure complete mixing at all times. Unmixed gravel was nowhere found, but in some places fine streaks of loam had been left unmixed with gravel, in spite of every effort to avoid it.

The gravel in the 100% gravel section was not rolled to a large extent and was not watered. When the gravel dump had reached the height of the trestle, the track was raised by grading up, until the full height of the dam was attained.

The rock for slope pitching was obtained from the rocky basalt bluff on the north side of the cañon, a short distance below the dam. It broke up in fragments from 1 cu. ft. in volume down. A part of it was loaded by an orange-peel excavator, but most was handled from wheel-barrows into dump-cars. It was hauled by rail, dumped on the slopes from the dump-cars, and sloped by hand. The total required was 36 000 cu. yd.

The construction of the auxiliary structures contained no elements of special interest.

The work on the installation of the plant was commenced in December, 1906. The first gravel was dumped in May, 1907, and the dam was completed on January 1st, 1908.

Shrinkage.—The design of the dam calls roughly for one-third of the mass 100% gravel, one-third 50% gravel, and the remaining third 67%, 75%, and 80% gravel. Laboratory tests indicated a shrinkage of about 10% for the various mixtures, and from this it was figured that there would be required 490 000 cu. yd. of gravel and 191 000 cu. yd. of loam, or a total of 681 000 cu. yd., to make the 637 000 cu. yd. of compacted dam.

The actual quantity of gravel excavated corresponds closely to the estimate, but the quantity of soil handled was 76 000 cu. yd. in excess of that figured. This large excess must be principally attributed to the difficulty of keeping the soil and gravel apart in the gravel pit, and may be partly due to the occurrence of volcanic ash or dust in the delivered soil, which may not have assisted in swelling the quantities. The proportion of loam in the gravel, where it came unavoidably mixed with gravel from the gravel pit, may have been under-estimated, and it is quite probable that much of the gravel, which from all appearances contained no soil, may have held proportions of from 5 to 10 per

FIG. 1.—GRAVEL BANK ON NORTH SIDE OF CAÑON, BELOW COLD SPRINGS DAM.

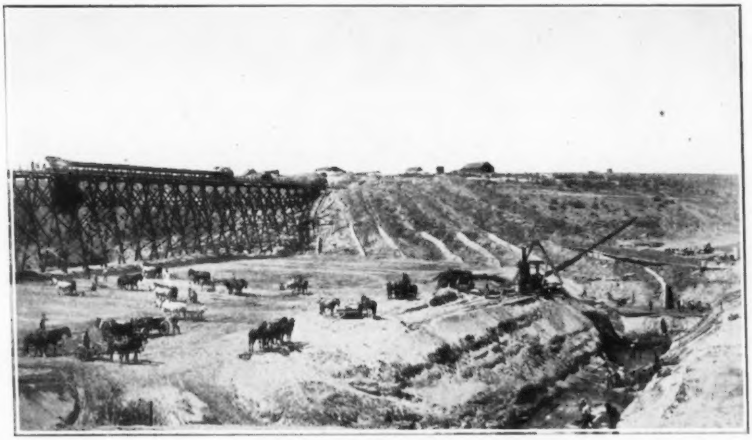


FIG. 2.—BASE OF DAM, CUT-OFF TRENCH, CONSTRUCTION TRESTLE, ETC., COLD SPRINGS DAM.

cent. While the excess soil has added to the cost, it can hardly be deemed injurious, as regards the drainage qualities of the gravel or its stability, and it has also added to its mass weight.

Cost.—The total cost of the dam, arranged by its principal features, is shown in the following tabulation:

Main dam		$364 140
Auxiliary structures:		
Inlet works	$16 140	
Outlet works	19 710	
Spillway	35 010	
		70 860
Preliminary engineering		5 000
Total, 49 000 acre ft., at $8.98 per acre ft.		$440 000

General administration, engineering and supervision, other than preliminary engineering, are included in the above figures.

The principal details of the cost of the main dam are as follows:

Embankment (yardage on basis of excavation measurement):			
Material from gravel pit: Gravel	490 000 cu. yd.		
Earth	100 000 " "		
	590 000 cu. yd.	at $0.385	$227 020
Material from borrow-pits, spillway, and trenches:			
Earth, 86 300 cu. yd., orange-peel excavator, 36.1 cents		$31 120	
80 700 " " wheel scrapers, 19.4 "		15 630	
167 000 cu. yd. average, at $0.28			46 750
Rip-rap:			
From quarry	32 500 cu. yd., at	1.46	47 480
" trenches	3 400 " " (charged to excavation).		
Excavation:			
Earth from trenches, spillway, etc., temporarily or permanently wasted	34 400 cu. yd.	0.293	10 060
Hardpan or loose rock	6 600 " "	1.37	9 020
Solid rock	4 000 " "	2.84	11 340
Concrete cut-off walls	327 " "	13.79	4 510
Drainage, including all work to July 1st, 1910			7 960
			$364 140

A further analysis of the two principal items may be of interest:

Embankment: Material from gravel pit, 590 000 cu. yd.

Steam-shovel excavation: Rail haul, average distance $2\frac{1}{4}$ miles, average rise 65 ft.

Cost per cubic yard, measured in excavation:

Steam shovel			3.6 cents.
Transportation:	Railroad operation	5.5 cents.	
	Track maintenance	1.7 "	7.2 "
Work on dam:	Scraping, spreading, and mixing	8.2 "	
	Sprinkling	0.2 "	
	Rolling	0.5 "	
	Water supply	0.7 "	9.6 "
Plant depreciation		10.4 "	
Plant maintenance		0.4 "	10 8 "
General supplies			1.8 "
Camp, shops, warehouses			1.2 "
Cleaning up, transfer of plant			0.6 "
Engineering, administration and general expenses			3.7 "
			38.5 cents.

Embankment: Material from borrow-pits, 86 300 cu. yd., orange-peel excavator, wagon haul, 500 to 2 000 ft.

Cost per cubic yard, measured in excavation:

Excavation		13.1 cents.
Wagon haul		6.9 "
Sprinkling		0.4 "
Rolling		0.7 "
Water supply		0.8 "
Plant depreciation	8.5 cents.	
Plant maintenance	0.4 "	8.9 "
General supplies		1.1 "
Camp, shops and warehouses		1.2 "
Cleaning up, transfer of plant		0.5 "
Engineering, administration and general expenses		2.5 "
		36.1 cents.

FIG. 1.—CUT-OFF TRENCH ACROSS VALLEY BOTTOM, COLD SPRINGS DAM.

FIG. 2.—TRENCH FOR OUTLET CONDUIT, SHOWING CHARACTER OF ROCK, COLD SPRINGS DAM.

The cost of gravel and earth for the main embankment, based on bank measurement, is higher than the foregoing figures indicate, by reason of heavy shrinkage, and is shown as follows:

	Excavation measurement.		
Material from gravel pit	590 000 cu. yd.,	at 38.5 cents	$227 020
" " borrow-pits, etc.	167 000 " "	at 28.0 "	46 750
Total	757 000 cu. yd.	at 36.2 cents	$273 770
Embankment measurement	637 000 cu. yd.	at 43.0 cents.	

The prices paid for labor and fuel were as follows:

Common labor	$1.60 to $2.40	for 8 hours.
Teamsters	2.40 to 2.60	" " "
Teamsters, with team	4.50	" " "
Steam-shovel engineers	6.20	" " "
Cranesmen	4.00	" " "
Locomotive engineers	3.60	" " "
Carpenters	$3.60 to 4.00	" " "
Coal, per ton, on work	8.63	

Behavior of Reservoir Since Completion.—Water was first admitted into the reservoir on February 19th, 1908, and reached its maximum height for that year on June 5th, at Elevation 584.4, and its minimum height on October 20th, at Elevation 561.8. Water was again admitted on November 5th, 1908. In 1909 it rose, on June 4th, to 604.1, and dropped, on October 15th, to 564.7. In 1910 it rose, on March 25th, to 616.2, and, on May 11th, to 616.7, standing at that time within 4.8 ft. of the spillway crest, with the reservoir containing 42 000 acre-ft.

From the time water was first admitted, measurements have been made of the inflow through the feed canal and from Cold Springs Creek at times of flood, of the outflow through the outlet canal, and of the evaporation and rainfall, which have yielded some interesting information as to seepage losses, as shown graphically on Plate XXXI.

Close watch for first evidences of seepage below the dam was also kept. The drain tile in the lower toe of the dam, serving the portion to the north of the outlet conduit and including the highest part of the dam, discharges into a small concrete basin in the old creek bottom, which is provided with a weir (Weir No. 1, on the plan of the dam, Plate XXX). The first water came from this drain

on April 2d, 1908, with the reservoir water at 571.5, or 27.5 ft. above the valley bottom. The maximum flow during 1908 was 0.17 sec-ft. During 1909 it was 0.26 sec-ft., and during 1910, 0.26 sec-ft.

In the flat below the toe of the dam, for a distance of about 1 000 ft., ground-water was observed to be gradually rising, and water began to ooze from the soil banks on either side of the meandering creek channel. This condition gradually extended downward for a distance of ½ mile. Weirs were maintained at two points in the creek channel, Weir No. 2 about 500 ft., and Weir No. 3 about 3 000 ft. down stream from the dam; and the flow, as measured, is shown on Plate XXXI.

It should be understood that the outlet canal skirts the south cañon side, and after leaving the rock adjoining the dam, runs through fine, sandy material, seepage below the hillside being evident in many places. This contributed to the flow, as measured in the creek weirs, to an extent which it has not been possible to determine definitely. The time during which the outlet canal was operated is shown at the bottom of the diagram, Plate XXXI, from which it will be noted that this flow is stopped in the fall of the year after the reservoir has been practically emptied; so that the effect of cessation of flow in the canal and that of the lowering of the water surface in the reservoir are combined in the reduced flows measured at the weirs. From the diagram it will be seen that the flow at Weir No. 2 reached maxima for the three successive years of 1.0, 1.26, and 1.46 sec-ft., while that at the lower creek weir shows maxima of 1.28, 3.27, and 3.03 sec-ft., respectively, these flows peaking about 4 weeks after the highest water occurred in the reservoir.

The portion of the dam to the south of the outlet conduit is served by a drain which discharges into the conduit. This did not show any flow during the years 1908-09, but during 1910 the flow was 0.08 sec-ft. during May, and, with the lowering of the water, this has again dried up. The lower end of this drain is at Elevation 575, or 29 ft. and 42 ft., respectively, below high water in 1909 and 1910. When moisture showed in the low places in the flat immediately below the dam in 1908, it was deemed advisable to lay additional drainage pipes, and, during the following winter, and also during the winter of 1909-10, a more complete system, consisting of a main 12-in. drain and cross-lines of 3-in. drains, was constructed, and has been entirely effective in maintaining the surface in a perfectly dry condi-

FIG. 1.—SPILLWAY, COLD SPRINGS DAM.

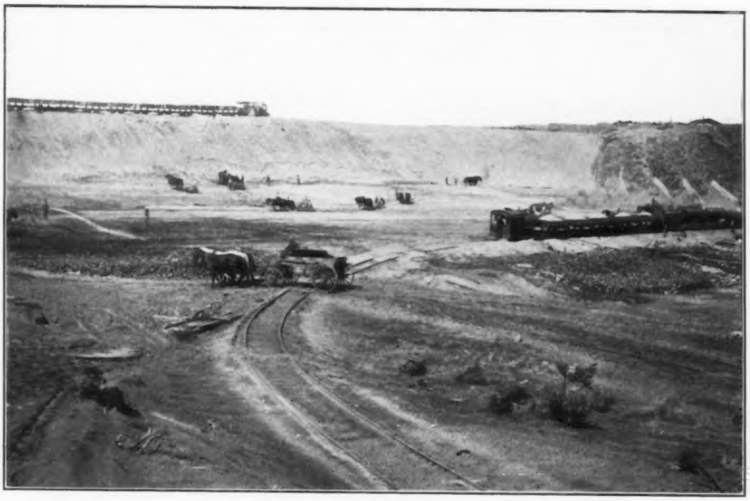


FIG. 2.—CONSTRUCTION TRESTLE BURIED IN GRAVEL DUMP, COLD SPRINGS DAM.

tion. Some drains have also been laid at the foot of the hillside below the outlet canal, to intercept seepage from this apparent source.

The course of seepage water from the reservoir is uncertain. It is not improbable that a small quantity of water passes through the body of the dam. The flow from the drain serving the highest portion of the dam carried about 0.25 sec-ft. with the reservoir water at 616.8, or 76.8 ft. above the valley bottom. The appearance of oozing water along the sides of the creek channel, for a considerable distance below the dam, indicates that seepage water may travel through seams in the underlying rock, or entirely around the outer slopes of the rock. It is significant that lower portions of the flat, separated from the dam by the meandering creek channel, show water well above the creek bottom, indicating a roundabout course of this seepage water.

The exposed rock surface of the bluff on the north side of the cañon immediately below the dam showed no seepage, until 1910, when a small spring made its appearance near the foot of the bluff, close to the toe of the dam, attaining a maximum flow during the season of 0.044 sec-ft. The interior of the outlet conduit was carefully examined, and while some moisture was in evidence, no dripping leaks were found where the conduit passes through the water-tight section of the dam. One small squirting leak, the size of a fine straw, was found near the lower end of the conduit where it passes through the 100% gravel section of the dam immediately above the highest cut-off wall, the pressure probably being due to the head accumulated by this wall, which makes tight connection with the tunnel rock trench in which the conduit was built.

As regards seepage under the dam on the south hillside, it has been stated that the maximum flow measured from the drain was 0.08 sec-ft. during 1910; the only further evidence of seepage along this part of the dam was a moist spot near the toe of the dam at about Station 17, where the elevation of the ground is about 600, or 17 ft. below the highest water in 1910. An investigation will be made as to whether this seepage may have been due to a break or obstruction in the drain pipe. Such break was found near Station 27 in the highest part of the dam in April, 1909, due undoubtedly to insufficient bedding of drain pipe. Just previous to this break, the drainage water from this line of tile, which had always been running crystal clear, showed a small amount of very fine sand, while the delivery

dropped from 0.25 to 0.19 sec-ft. Repairs were made during the following fall, at a time of low water in the reservoir. The down-stream slope of the dam has at all times been perfectly dry, nor is it conceivable that water would show at an elevation higher than the ground adjoining.

The diagram on Plate XXXI is especially interesting in that it shows the reservoir seepage losses determined by striking a balance between measured inflow and precipitation on the one hand, and measured outflow and evaporation on the other. The rate of loss at first filling was about 30 sec-ft., which quickly dropped to 20 sec-ft., and then continued to drop slowly during the first season until a slight return flow was measured. The maximum rate of loss during 1909 was 17 sec-ft., when the submerged area measured 1 003 acres, and in 1910 this rate rose to 34 sec-ft., at which time the water-covered area in the reservoir was 1 310 acres.

The amount of seepage during filling is dependent not only on the area submerged and the voids in the underlying soils, but also on the rate of filling. The higher this rate, the greater will be the new area submerged in a given time, and the steeper will also be the water gradient in the adjoining area. It is possible that this fact may account for the apparent anomaly of the seepage curve peaking several weeks prior to the dates when highest water in the reservoir was reached. The rate of filling is indicated by the steepness of the capacity curve, and it will be noted that the summits of the seepage curve coincide fairly with the breaks in the upward slope of the contents curve, showing a reduction in the rate of filling.

The dependence of seepage on the rate of filling is also indicated by the marked difference in seepage for the same water elevations under conditions of filling and of emptying the reservoir. For instance, on August 10th, 1909, return flow, during emptying, commenced with the reservoir water at 589, while previously, on February 26th, 1909, with water at the same elevation, the seepage loss during filling was 14 sec-ft. Similarly, in 1910, the return flow commenced on August 27th, with the reservoir emptying and with the water at 591, while on December 30th, 1909, with the water at the same elevation and the reservoir filling, the seepage loss was 19 sec-ft.

The totals of the losses from the reservoir for various periods are shown in Table 2.

FIG. 1.—TOP AND DOWN-STREAM SLOPE, COLD SPRINGS DAM.

FIG. 2.—UP-STREAM FACE AND GATE-TOWER, COLD SPRINGS DAM.

TABLE 2.—LOSSES FROM COLD SPRINGS RESERVOIR.

Period.	Total loss, in acre-feet.	Evaporation loss, in acre-feet.	Seepage loss, in acre-feet.	Maximum area submerged, in acres.
February 19th, 1908				
	6 717	2 384	4 333	659
November 1st, 1908				
	8 311	4 254	4 057	1 060
November 1st, 1909				
	14 916	5 122	9 794	1 836
November 1st, 1910				
	29 944	11 760	18 184	......

The evaporation loss was determined by applying the evaporation, measured each week in a floating pan, to the average reservoir surface for that week. This method leads to too small results, because the evaporation proceeds from the marginal areas as well as from the water surface, and because the actually exposed reservoir surface at times of wind, when evaporation is most rapid, is increased by the formation of waves to a far greater degree than is the case in a pan. Therefore, the resultant seepage losses as given are correspondingly too large, the writer having been unable to find a satisfactory basis for applying corrections.

The only evidence of escaping seepage water was in the creek channel below the dam, which averaged, as measured on Weir No. 3, for the three periods given in Table 2, 0.7, 1.4, and 1.8 sec-ft., respectively, resulting in an aggregate visible flow of 2 630 acre-ft. Assuming that one-fourth of this may be charged to outlet canal loss, 1 970 acre-ft. may have come from the reservoir as visible flow and the remainder, 18 184, less 1 970, or 16 214 acre-ft. of reservoir seepage must have been stored in the ground during the period of 2 years and 10½ months. Comparing this with the maximum area covered (1 386 acres), the loss is about 11.8 ft. in depth per acre, sufficient to fill soil having 45% voids to a depth of 26 ft. As the depth to bed-rock may average more than this, and especially as the water movement must be largely horizontal, the losses above recorded do not seem surprising. It is probable that losses of this character will continue for several years, until some average water-table in the surrounding soil shall have been established, after which the loss during the early season will be largely offset by return flow on emptying the reservoir.

Settlement.—The dam was finished with a super-elevation of 2% of the height, as previously stated, to take care of possible settlement. Bench-marks established on the concrete bridge piers at the time of construction and at points on the water slope where the dam is highest, show a settlement to date of about ¼ per cent. A series of bench-marks was established on top of the dam on January 2d, 1909, after a depth of 41 ft. of water had stood against the dam during the previous season. These bench-marks show a subsequent settlement, varying from 0.1 to 0.45% of the height, the maximum settlement at any point being 0.366 ft., where the dam is 84 ft. high.

The dam design was approved by the Director and the Chief Engineer of the Reclamation Service, and by A. J. Wiley, M. Am. Soc. C. E., as Consulting Engineer. The designs were prepared and the work executed under the general supervision of Mr. E. G. Hopson, Associate Supervising Engineer, and the writer. The project was in immediate charge of John T. Whistler, M. Am. Soc. C. E., and the engineering and construction work at the dam was in charge of Mr. B. H. Davis. For the last two years the project has been in charge of H. D. Newell, Assoc. M. Am. Soc. C. E., who has directed the later work and measurements.

Conconully Dam.

This dam is lower than the Cold Springs Dam and contains less material, but is of comparative interest as it was built by the hydraulic process. It is located in Northern Washington, on Salmon Creek, a branch of the Okanogan River, which empties into the Columbia River. The dam is part of the Okanogan Project of the United States Reclamation Service, intended for the irrigation of 10 000 acres of land along the west bank of the Okanogan River, between the Towns of Okanogan and Riverside. The land lies in benches above the Okanogan River, which has too flat a grade to serve as a source of supply. The water, therefore, is derived from the Salmon Creek, and as the midsummer flow is very small, the storage of flood flow was necessary.

Part of the needed storage (1 800 acre-ft.) is found in utilizing Salmon Lake, situated in a side cañon just above the main reservoir. The main storage was created by the Conconully Dam, the principal features of which are as follows:

Greatest height above bottom of creek channel	66 ft.
Width of valley on center line of dam.	815 "
Length of crest of dam	1 010 "
Top width	20 "
Length of spillway	180 "
Up-stream slope: upper portion $2\frac{1}{2}:1$, lower portion	$3:1$
Down-stream slope	$2:1$
Volume of dam	351 500 cu. yd.
Capacity of reservoir above outlet gates	13 000 acre-ft.
Required capacity of outlet works	100 sec-ft.
Area of reservoir at high-water line	460 acres.
Drainage area above dam	113 sq. miles.

The dam is intended to store the waters of the creek on which it is located, and is therefore independent of a feed canal. The drainage area above the dam is of distinctly mountainous and forested character, much of it being in forest reserve. The flow from Salmon Creek varies from 10 to 20 sec-ft. during 8 months of the year. The flood months are April, May, June, and July, during which the flow usually ranges between 100 to 300 sec-ft. The maximum flow recorded since 1904 is 577 sec-ft.

Reservoir Site.—The dam is 15 miles above the mouth of Salmon Creek and immediately below the junction of the two forks called the North and the West Forks, as shown on Fig. 3. The water in the reservoir fills a basin with a high-water area of 460 acres, the water backing up to the limits of the Town of Conconully.

The reservoir is surrounded by mountains on all sides except to the east where an extensive gravel bar exists, which is probably a glacial moraine. Without this bar, the water from the reservoir would flow in an easterly direction down Scotch Creek. The bar at its southern extremity rests against the mountain slope which farther down forms the east abutment of the dam. The width of the bar at the spillway crest elevation is about $\frac{5}{8}$ mile. On its eastern slope small springs exist, which give rise to Scotch Creek. Water first appears at an elevation above that of Salmon Creek on the west side of the bar, although

most of the springs are at a lower elevation. It is possible that these springs may result in part from seepage from the creek, but it is considered more likely that they find their origin in drainage from the mountain side to the south. In considering the possibility of seepage

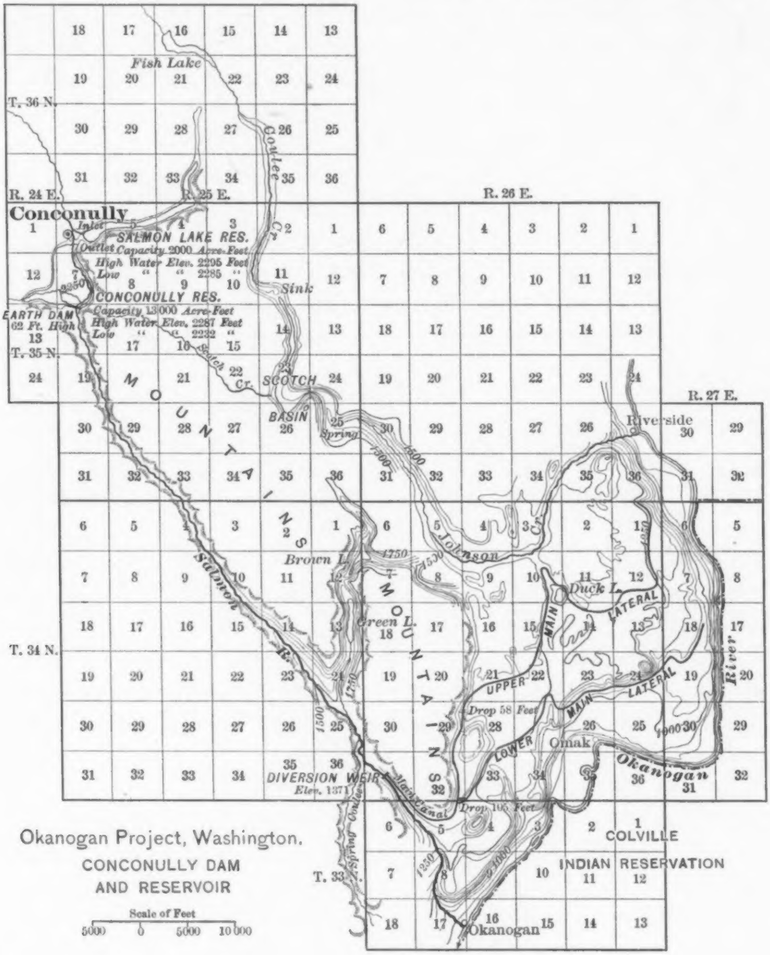


FIG. 3.

through this bar, from the rise of water in the reservoir, it was found that the average water grade with reservoir full would be about 1:165, and, aside from the fact that the reservoir side of the bar is blanketed with mixed material (including some surface loam), and that the

gravel is principally made up of sand, it was believed that the slight water slope precluded any possibility of material losses.

The flow of Scotch Creek has been measured to determine possible increases, and if any should take place, it will be easily possible to divert it for the irrigation of the lower lands. The reservoir water is released into Salmon Creek and picked up at a diversion dam about 11 miles down stream.

The reservoir bottom consisted of meadows, and was covered with thick brush, which has been slashed and burned. The surface soil is a black sandy loam for a depth of 4 or 5 ft., below which the sand gradually becomes cleaner, deeper layers occasionally showing coarser material, the rock being at unknown depth both in the reservoir bottom and at the dam site, where soundings were continued to a depth of 95 ft.

The dam is at the outlet of the valley, where it contracts to a width of about 1 200 ft., the gap being further reduced by a low mountain spur from the west, leaving an opening, about 1 000 ft. long, to be closed by the dam, the spur serving the purpose of a spillway by being cut down for a width of 180 ft. to the proper elevation.

The formation of the hillsides is a close, heavy granite, but in the above mentioned spur this granite merges into limestone of a rather soft and seamy character.

Design of Dam.—The total height above the valley bottom to which the water was to be raised is about 50 ft. The foundation could not be made water-tight, and the design simply aimed at a retardation of the percolation by providing a broad base to the dam due to flat slopes, and by driving a row of sheet-piling, running across the valley, joining the rock slopes at both ends.

Materials Available.—The following materials were available for dam construction:

Fine sandy loam near the surface, in the valley bottom, principally to be found up stream from the dam;

Gravel and sand from the gravel bar to the east of the reservoir, at a distance of from 2 000 to 5 000 ft.;

Talus material on the west mountain side, just below the dam, consisting of sand and silt from the disintegration of the granite rock, mixed with angular rock fragments of sizes from a man's fist to a cubic yard.

The latter material was selected as that most suitable for the dam, in connection with the method of construction to be followed.

The location of the dam is quite remote, the nearest railroad station being Wenatchee, Wash., 105 miles distant, from which material can be shipped by river steamer to Brewster, whence the wagon haul over mountain roads to the work is 45 miles.

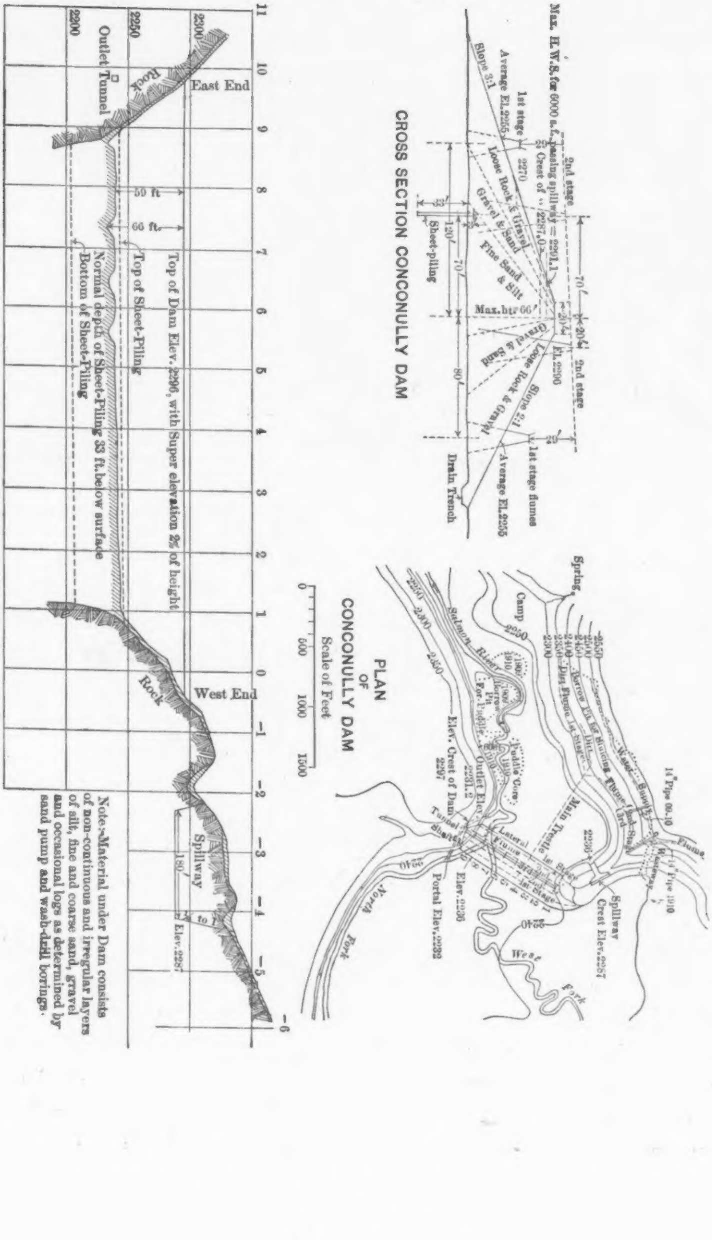
The country affords practically no local labor, and for machine repairs it would be necessary to send to Spokane, Seattle, or Portland. Thus it was desirable to depend as little as possible on mechanical equipment, and to utilize the local resources to their maximum extent. The use of the valley bottom material would have implied the installation of pumping machinery, in connection with either hydraulic or mechanical means of transportation, as the ground-water was close to the surface. Gravel and sand, obtainable from the gravel bar on the east side of the reservoir, would have necessitated the importation of a steam shovel, and probably railroad equipment. The material available on the steep mountain side above mentioned could be had at practically any elevation, and could be moved by the hydraulic process. Water was available in the West Fork of Salmon Creek by diversion through a 3-mile flume, the creek flow being 50 sec-ft. and more during April, May, and June, and gradually dropping to 10 sec-ft. in August and September.

The pit material, as revealed by numerous test-pits, appeared to contain a sufficient proportion of silt for producing a tight core. Consequently, the hydraulicking process was adopted, and the dam was designed in accordance therewith, the plan and sections being shown on Fig. 4.

The character of the material hydraulicked in 1908 is best shown by the physical analyses (Table 3) after from 10 to 40% of material coarser than would pass a 7 by 13-in. screen had been eliminated.

Much of the material hydraulicked in 1909 and 1910 was coarser than that from the 1908 pit No. 2. On the whole, the material was found to be far coarser than had been inferred from test-pit examinations.

It was considered desirable to place the core section as near forward in the dam as practicable so as to have it backed by the maximum quantity of more open material. As a result, the central plane through the core has a down-stream inclination. It was ex-



LONGITUDINAL SECTION, CONCONULLY DAM.

FIG. 4.

pected that no difficulty would result from this position of the core, as it would be possible to keep the down-stream dumps at a higher elevation than those up stream and thus maintain the central pond at a point well forward toward the reservoir side.

TABLE 3.—PHYSICAL ANALYSIS OF MATERIAL AVAILABLE FOR CONCONULLY DAM.

Pass sieve.	Stop on sieve.	Percentage, by Weight: Pit No. 1. Average of three samples.	Pit No. 2. Average of three samples.
7 by 13	2½-in.	15.8	40.65
2½	No. 4	27.2	32.69
No. 4	10	12.4	6.21
10	20	11.9	3.73
20	40	3.8	1.46
40	60	14.5	5.79
60	80	3.9	2.07
80	100	2.4	0.97
100	200	6.1	4.45
200	....	2.C	1.98
		100.0	100.0

The core section connects with the side-hill by cleaning to bed-rock and excavating a rock trench in line with the inclined central core plain. A drainage trench was provided at the down-stream toe of the dam, filled with coarse material, hydraulicked in, connecting with the old creek channel below the dam.

The spillway has a crest length of 180 ft. and a crest height 9 ft. below the top of the dam, giving an approximate overflow capacity of 4 500 sec-ft., with 5 ft. freeboard on the dam, or about 40 sec-ft. per sq. mile of drainage area. It was cut into the west ridge and was provided with concrete crest and channel lining, the point of discharge at the foot of the hill being 300 ft. from the down-stream toe of the dam.

The discharge from the reservoir is through a tunnel in the hard granite mountain side on the east, partly lined with concrete, in which two 36-in. common water-works gates were installed. These are set side by side, and operated through a vertical shaft connecting with the surface and well up stream from the center line of the dam. The tunnel was left unobstructed, and the water stored during the 1909 season was controlled by a temporary wooden gate in the shaft, the

FIG. 1.—SHEET-PILING AND FIRST-STAGE FLUMES, CONCONULLY DAM.

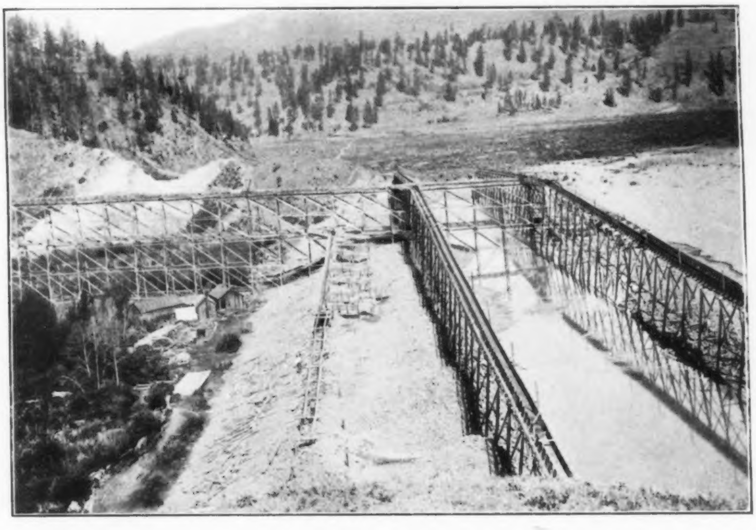


FIG. 2.—SECOND-STAGE FLUMES AND CENTRAL POND, CONCONULLY DAM.

final gates not being installed until the summer of 1909. The required discharge capacity is 100 sec-ft.

Construction.—Construction was commenced in the summer of 1907, during which year the following work was done:

- Building 3 miles of feed-water flume, mostly on steep mountain sides;
- Building a dirt flume, partly on mountain side, partly on trestle;
- Driving and jetting 855 ft. of 6 by 12-in. triple-lap, tongued and grooved, sheet-piling, 36 ft. long, for a distance of 33 ft. into valley bottom;
- Excavating 395 ft. of 8 by 9-ft. outlet tunnel, partly lined, through the east mountain side, and excavating a vertical shaft;
- Partial excavation of the spillway gap in the ridge at the west end of the dam.

During the season of 1908, 97 000 cu. yd. of material were sluiced from the borrow-pits, and the spillway excavation was completed and lined with concrete. At the end of the season, second-stage flume trestles were erected.

During the season of 1909, 188 000 cu. yd. of material were sluiced from the borrow-pits, and the permanent gates were installed in the outlet tunnel.

During the season of 1910, the remaining 64 000 cu. yd. of material were sluiced, mostly from the second-stage, and partly from the third-stage flumes. The dam was completed during August, 1910.

The building of auxiliary structures presented no special points of interest, and the following remarks will be confined to the hydraulicking feature.*

The original supply flume had a capacity of 17 sec-ft. Its location is shown on Fig. 5. At the end of the 1908 season it was decided to increase this capacity to 26 sec-ft. Small storage reservoirs were built above the flume intakes to permit of concentration of the flow during the dry season for two shifts or one shift each day. The large boulders found in the pit had to be broken by blasting, or wasted, to prevent them from accumulating in the bottom. For this reason two pits were kept in operation alternately.

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* A description by the writer of the details of the hydraulicking work during the year 1908 was published in *The Engineering Record*, April, 1909.

The feed-water was led down the mountain side through 14-in. No. 16, steel, slip-joint pipes, one line of pipe for each borrow-pit, and was used partly by the giant, which consumed from 1½ to 5½ sec-ft., through nozzles changed from 2 to 3½ in. in diameter, as required. The water was supplied under a pressure due to the difference in elevation of the feed-flume and the bottom of the pit, which difference ranged from 129 to 169 ft. for the first stage, and from 114 to 140 ft.

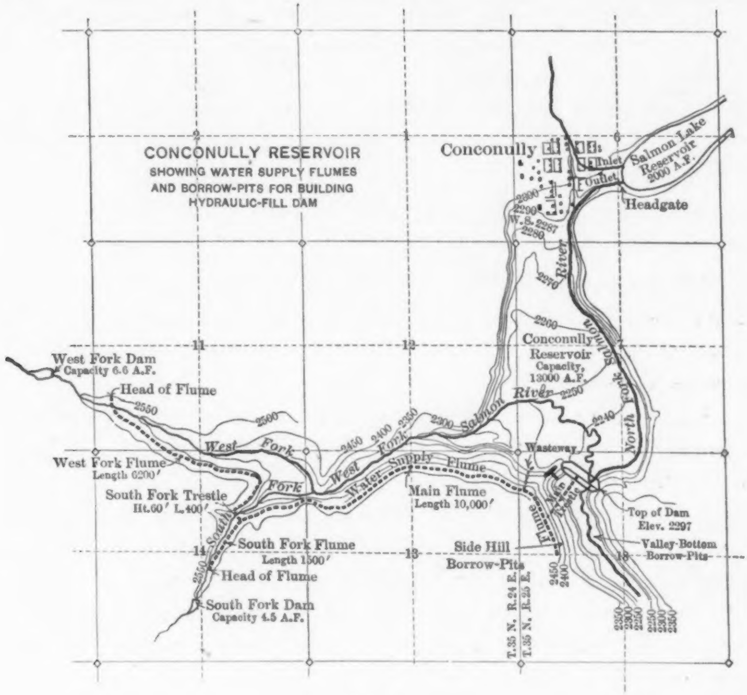


FIG. 5.

for the second and third stages. A flow of from 2 to 3 sec-ft. was delivered under pressure through a 4-in. pipe at the head of the borrow-pit dirt flume near its bottom, serving as push-water. The remainder of the available water was used as push-water at the point where the pit flume dropped its load into the main dirt flume. A small quantity of water, however, was allowed to enter the pit at its upper end on a level with the supply flume, in order to cause the fine upper material to slide in from above.

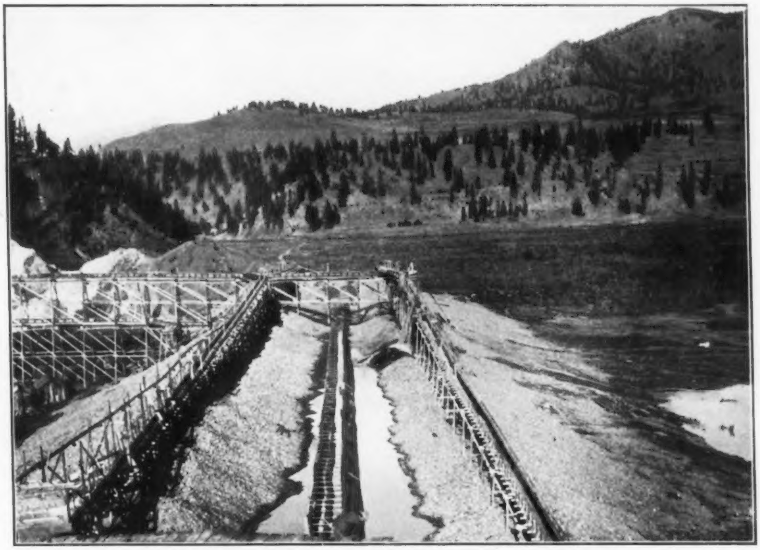


FIG. 1.—SECOND-STAGE FLUMES AND ARTIFICIAL CORE, CONCONULLY DAM.

FIG. 2.—THIRD-STAGE FLUMES, CONCONULLY DAM.

A 7 by 13-in. screen was used at the head of the pit flume during the 1908 season, to exclude large rock, but its use was discontinued after the water supply was increased.

The main dirt flume ran along the lower edge of the pits opposite the point on the dam equidistant from its ends, and then proceeded on a high trestle from the mountain side to the dam. On reaching the dam, the main flume connected on either side with two lateral flumes near the down-stream toe of the dam, and continued to similar flumes close to the up-stream toe. When the dam was built up to the elevation of the first lateral flumes, the main trestle was raised 29 ft., and was connected with a new flume laid along the mountain side, while the new lateral flume trestles were built closer to the center line of the dam. In the final finishing of the dam, a single flume was built on trestle near the center line.

The main dirt flume was built with wooden sides slightly inclined outward, and with a curved bottom of No. 10 mild steel with a 12-in. radius. The width at the top was 2 ft. 9 in., and the total depth 2 ft. 3 in. The velocity of the water ranged from 14 to 18 ft. per sec.

It soon became apparent that the angular rocks sliding on the bottom caused serious wear, and when 11 000 cu. yd. had been delivered, many holes had been worn within a strip in the center 6 in. wide, the steel higher up showing little wear. The flume was then given a flat wooden bottom 16 in. wide, and lined with No. 10 mild steel, which stood the wear far better.

At the end of the first season, when it was decided to increase the water supply from 15 to 26 sec-ft., the dirt flume was rebuilt to rectangular shape, with a bottom 30 in. wide and lined with ¼-in. high-carbon steel, and 27-in. sides lined for the lower 6 in. with No. 10 mild steel. The heavier and harder steel answered the purpose satisfactorily, and lasted through the delivery of 252 000 cu. yd. of material, showing serious wear only at the butt joints.

The flumes had 4% grades, except the short borrow-pit flumes, which had 8% grades, and the third-stage flumes for the finishing of the dam, which for part of the distance back had a 3% grade.

The material was discharged from delivery points at the dam in two rows of cones, forming ridges, the principal ridge being along the down-stream slope. By deflecting screens, gratings, spouts, and other means, the coarsest material was discharged, as far as possible, out-

ward, and the finer material inward, toward the pond maintained between the two ridges. The surplus water from the pond was drawn off on the reservoir side through flumes near the ends of the dam, which were alternately raised 6 in. at a time, the pond being maintained at a depth of from 12 to 18 in.

The material settling in the pond consisted of very fine sand and silt, the coarser sand and gravel coming to rest on the sloping sides of the pond, and the large rock dropping vertically and sliding down the cone slopes. The pond at first was quite wide, but, as elevation was gained, it narrowed up to such an extent that in spite of skillful handling, the sloping coarse sand layers would at times extend well into the puddle section and sometimes clear across it. Such layers were broken up by systematic stirring with paddles, but when this tendency to stratification became more marked and could not be satisfactorily prevented or counteracted, it was decided to introduce an artificial core with puddling material from other sources. The surface material in the valley below the dam, consisting of black, loamy sand, was well suited to form a core. This material was hauled in by scrapers on an up-hill road, dumped on a platform and washed into the dam through an 8-in. pipe. In order to insure against stratification across this core, two wooden diaphragms were built of 2 by 4-in. studding and 1-in. boards, which were first given a vertical position and made to step back in sections, so as to have their center plane correspond, as nearly as possible, with the center plane of the general core, but which were later built in a sloping position. Thus, while the material from the upper borrow-pits was hydraulicked in on the slopes, the core material was washed in through the 8-in. pipe between diaphragms. The artificial core was started at an elevation 14 ft. above the general base of the dam in the late spring of 1909, and was continued about 39 ft. up to the high-water line. It contains in the aggregate 11 600 cu. yd., and, owing to the long haul of about 1 000 ft. on a 7% up-grade, and also to the necessity of using a large quantity of lumber for diaphragms, its cost was quite high.

The coarse rock, as dumped on the outer slopes, was of sufficient size to serve as rip-rap, but it did not prove possible to deposit it to final slopes by the use of water alone, and after hydraulicking was completed, it required a large amount of hand-work to obtain reasonably good slopes.

FIG. 1.—SIDE-HILL BORROW-PIT, CONCONULLY DAM.

FIG. 2.—MATERIALS BEING DUMPED FROM FLUME, CONCONULLY DAM.

As the quantity of rock found in the borrow-pits was larger than had been estimated, the relative quantity of rock on the water slope became sufficient to justify the steepening of this slope from 3 : 1, as had been originally designed, to 2½ : 1 for the upper 26 ft. of the dam, as shown by the dotted lines in the central section on Fig. 4.

The dam was built with a super-elevation of 1 ft. across the valley, equivalent to nearly 2% of its height. In view of the hard pounding and washing which the material received in being dumped, and the prevalence of sand and gravel, this provision may seem excessive. During the progress of construction, however, it was found that as the load came on the base, considerable settlement occurred in the trestles, due apparently to a compacting of the fine loamy sand in the foundation. The amount of this settlement, extending over the last 560 days of construction, is shown on Fig. 6 by contour lines, the maximum settlement being 3.9 ft., and occurring along the old creek bed. In total volume, this settlement amounted to 15 500 cu. yd., and while it may have ceased on the completion of the dam, it was deemed wise to finish to the super-elevation above mentioned.

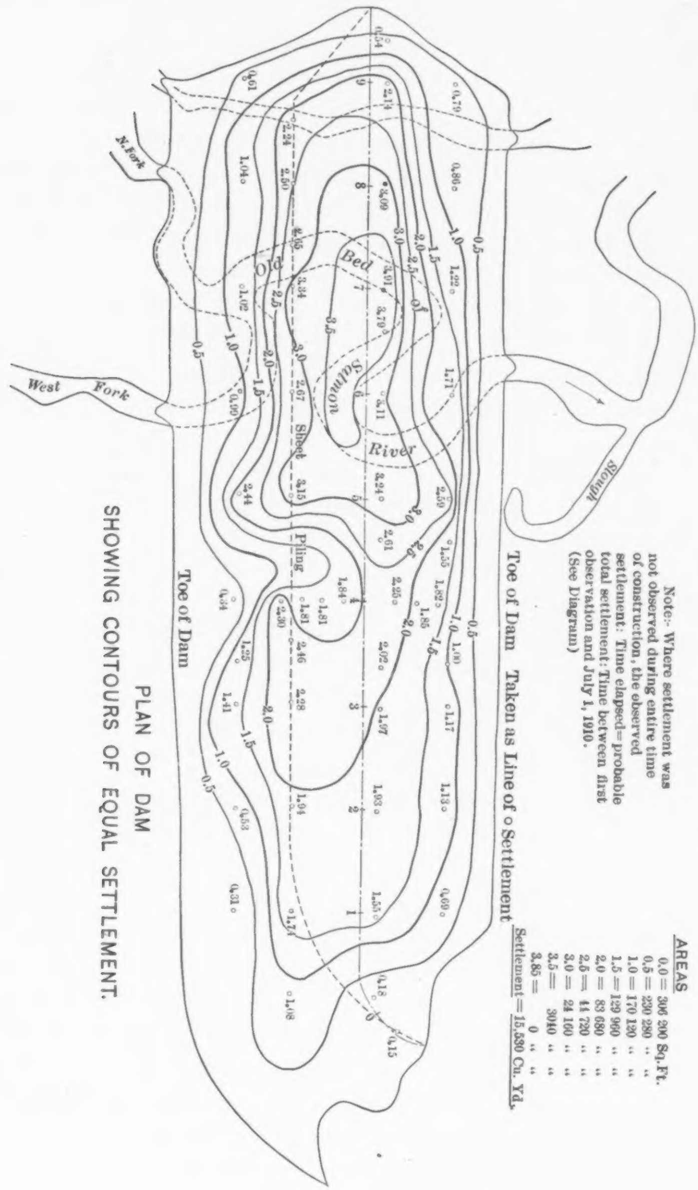
Careful watch was kept for possible evidence of a swelling up of the ground surface beyond the toes of the dam, but none was observed.

The degree of effectiveness of the sluicing for each season is shown in Table 4.

TABLE 4.—SLUICING CONCONULLY DAM.

Season.	Hours actually sluiced.	Hours lost.	Average water supply.	Cubic yards from borrow-pits.	Average cubic yards per hour sluiced.	Average percentage of material carried.
April 22d-October 15th, 1908...	1 935	231	13.2	97 165	50.2	2.85
" 14th-November 13th, 1909.	2 892	564	20.2	188 300	65.1	2.42
March 31st-June 24th, 1910......	1 154	297	22.3	64 900	55.4	1.87
	5 981	1 092	....	......	....	....

The decreasing percentage of material carried by the water, in spite of increasing skill and larger water supply, is due to the increasing proportion of coarse rock in the borrow-pits. It will be observed that, of the total 7 073 hours of sluicing, 1 092 hours, or about 15%, were lost. This was due principally to clogs in the dirt flumes, and in part also to breaks in the feed flume, and to the necessity of replacing the dirt flume bottoms.



PLAN OF DAM
SHOWING CONTOURS OF EQUAL SETTLEMENT.

FIG. 6.

The largest output in any one day was 181 cu. yd. per hour, at which time the water carried 5.3% of solids. The largest rocks carried in the flume, with 25 sec-ft. flow, weighed 250 lb.

The waste-water from the pond did not run out perfectly clear, but contained from 0.1 to 0.2% of silt, which sometimes rose to 0.5 per cent. The total loss of silt from this cause during the entire work is estimated at 20 000 cu. yd.

The segregating effect of the water caused an increase of dam measurement over borrow-pit measurement, possibly offset in part by a shrinkage where sand, gravel, and rock, with some silt, happened to be well mixed in the dump pile. The swell in volume during the first season was estimated at 12%, but, for the completed work, a lower percentage is computed, as follows:

Sluiced from side-hill borrow-pits.......		349 455 cu. yd.
Loss in waste-water..................		20 000 " "
Remaining in dam, pit measurement....		329 455 cu. yd.
Volume of dam above original surface....	336 000 cu. yd.	
Volume of dam below original surface....	15 500 " "	
Total volume of dam...	351 500 cu. yd.	
From valley pits........	11 600 " "	
From side-hill pits, bank measurement	339 900 cu. yd.	
Swell, 3.2%.........................		10 445 cu. yd.

Many physical analyses were made of the material in the tight section of the dam, of which those in Table 5 are representative.

Table 6 contains the results of a few percolation experiments which were made.

The temperature was not noted continuously, but was subsequently reported to have averaged approximately 70° Fahr. A correction has been applied on the basis of Hazen's formula, and the figures in Table 6 are on a basis of 50° temperature.

TABLE 5.—PHYSICAL ANALYSES.

Pass sieve No.	Stops on sieve No.	PERCENTAGE, BY WEIGHT.					
		A	*B*	*C*	*D*	*E*	*F*
....	4	0	0	8.45	6.40	0	0
4	10	0.52	0.16	6.70	9.38	0.56	0
10	20	1.03	0.37	11.15	16.60	3.31	0
20	40	0.76	2.34	23.59	12.26	7.08	0.50
40	60	7.68	10.66	15.52	43.45	10.16	0.58
60	80	12.21	8.16	9.65	5.75	12.69	0.84
80	100	7.70	5.83	2.41	1.73	6.25	0.58
100	200	33.70	36.99	12.60	2.93	29.14	19.00
200	....	36.48	35.49	9.93	1.50	30.86	78.50
		100.0	100.0	100.0	100.0	100.0	100.0
Taken from dam at Station............		5 + 50	Average of 5 samples	Average of 3 samples	9 + 00	3 + 80	1 + 10
Elevation...........		2 252	2 243	2 251	2 252	2 269	2 269
Source..............		Side-hill borrow-pits.				Valley borrow-pits.	
		Pure silt.			Coarse sand found stratified in core.	Fine, black, sandy loam.	

TABLE 6.—PERCOLATION EXPERIMENTS, CONCONULLY DAM.

Sample.	Percolation rate, in gallons per acre per day: 3 ft. of material under a 3-ft. head.	Rate observed at end of following period.
MATERIAL DEPOSITED IN DRY, LOOSE CONDITION IN TANK.		
A	561 000	12 hours.
D	16 600 000	¾ "
E	98 000	23 "
F	13 300	62 "
MATERIAL PACKED IN TANK IN 8-IN. LAYERS WITH IRON CONCRETE TAMPER.		
E	34 700	39 hours.
F	4 500	12 "

The high rate of percolation in the fine silt, Sample *A*, is surprising, even though the material was not compacted in the tank except in so far as it settled by subsequent admission of water. Samples *D* and *E*

CONCONULLY DAM AND RESERVOIR COMPLETED.

showed one-third the rate of percolation when tamped, as compared with being put in loose, and if this ratio be likewise applied to Sample *A*, a rate might have been noted of nearly 190 000 gal. per day.

The sandy loam, represented by Samples *E* and *F*, may be considered water-tight. Sample *E* was taken closer to the point of discharge than Sample *F*, and averaged coarser, which explains the high rate of percolation.

Referring again to core silt, as per Sample *A*, the effect of compacting by superincumbent weight would probably reduce its rate of percolation, there being no apparent reason why this material, when well packed and with surplus water squeezed out, should not be as tight as clay.

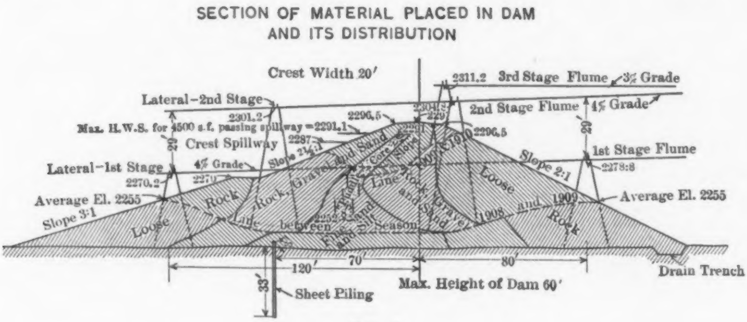


FIG. 7.

The results with Sample *D*, which was taken from a coarse sand layer which had penetrated the silt core, shows the absolute necessity of avoiding stratification, and justified the additional expense involved in the adoption of the artificial core.

The distribution of the material in the dam as built is fairly represented by the cross-section, Fig. 7, which also shows the stages of completion at the end of each season, and may with advantage be compared with the central section on Fig. 4, showing the original design.

Cost.—The total cost of the reservoir, arranged by its principal features, is shown in Table 7.

The items in Table 7 include all charges for administration, engineering, and general expenses. The cost of puddle core includes lumber for diaphragms.

The total material in the dam is 351 500 cu. yd. and the combined cost of hydraulicking and puddle core is $178 522, making the average

cost, on the basis of bank measurement, 50.8 cents per cu. yd., including lumber for core and all overhead charges, but exclusive of the cost of sloping.

TABLE 7.—COST OF CONCONULLY DAM.

	Hydraulicking.	Main dam.	Total.	Cost per Cubic Yard, Bank Measurement.	
				Side-hill pits, 339 900 cu. yd.	Bottom pits, 11 600 cu. yd.
Clearing reservoir site			$7 652		
Main dam:					
Creek diversion		$965			
Clearing dam site		936			
Trenches		1 862			
Sheet-piling		13 982			
Hydraulicking:					
Plant	$69 099			20.33 cents.	
Supplies	11 783			3.47 "	
Labor	74 755			21.99 "	
Total		155 637		45.79 cents.	
Puddle core		22 885			$1.97
Sloping		8 465			
Cleaning up					
Miscellaneous		1 382			
			208 219		
Outlet works			23 114		
Spillway			34 613		
Telephone			2 827		
Real estate			30 977		
General investigation of dam sites			19 028		
13 000 acre-ft., at $24.95			$324 325		

The details of the cost of hydraulicking are shown in Table 8.

The prices paid for labor were as follows:

Common labor	$2.25 to $2.50	for 8 hours.
Pitmen	2.75 to 3.00	" " "
Giant men	3.00	" " "
Powder men	3.00	" " "
Carpenters	4.00 to 4.50	" " "
Foremen	5.00	" " "

It will be noted that the steel lining cost 4.25 cents for material, and 1.48 cents for replacing, or a total of 5.73 cents per cu. yd. of sluiced material in the dam.

Results.—During the season of 1909 after danger from flood was past, water was allowed to rise against the dam to Elevation 2 253.3, or 17 ft. above the valley bottom. As the dam was still unfinished with the opening of spring in 1910, it was necessary to observe great

TABLE 8.—COST OF HYDRAULICKING, CONCONULLY DAM.

PLANT.

	Totals.	Cost, in cents per cubic yard.
Feed supply dams and flumes	$17 140	5.04
Dirt-flumes and trestles, exclusive of flume lining	20 128	5.94
Steel lining	14 964	4.40
Pipes, giants, and hose	3 725	1.09
Electric light plant	1 250	0.37
Proportionate share of camp buildings	1 116	0.33
Superintendence	5 076	1.49
Administration, engineering, and general expenses	7 805	2.29
	$71 204	20.95
Less value of plant on hand	2 105	0.62
	$69 099	20.33

SUPPLIES.

Tools	$2 150	0.63
Rubber boots and clothing	2 700	0.80
Powder and explosives	4 793	1.41
Proportionate share of camp buildings	170	0.05
Superintendence	776	0.23
Administration, engineering, and general expenses	1 194	0.36
	$11 783	3.48

LABOR.

	Totals.	Cost, in cents per cubic yard, bank measurement.
Foremen	$4 274	1.25
Building road to pit	571	0.17
Clearing borrow-pit	547	0.16
Feed-supply flume tenders	7 261	2.14
Giant men	2 365	0.70
Pit men	13 232	3.89
Clearing pit of rock	8 358	2.46
Building lateral flume in pit	963	0.28
Hauling and laying pipe in pit	2 707	0.79
Dirt flume tenders	7 748	2.28
Labor, steel lining	4 975	1.46
Spreading material and puddling in dam	1 354	0.40
Carpenters on dam and flumes	3 915	1.15
Blacksmith	944	0.27
Operating light plant	767	0.24
Transportation laborers	1 308	0.38
Dismantling plant	936	0.28
Proportionate charge for camp buildings	998	0.29
Superintendence	4 543	1.34
Administration, engineering, and general expenses	6 989	2.06
	$74 755	21.99

caution in permitting storage until the work had reached spillway height elevation. Thus, storage could not be commenced at an early date, and the reservoir was not filled during 1910 to an elevation higher than 2 271.5, or about 33 ft. above the valley bottom, and about 15.5 ft. below spillway height.

A small spring existed at the down-stream base of the spillway ridge, midway between the west end of the dam and the discharge end of the spillway, which had been kept under close observation, it being deemed probable that it might be supplied from ground-water from the up-stream side of the spillway ridge. This view was confirmed when the flow increased during the 1909 season from its previous steady flow of 0.115 sec-ft., to 0.21 sec-ft. as the water rose 17 ft. above the valley bottom, after which it dropped back to its original amount as the reservoir was gradually emptied.

During 1910 the water rose to 33 ft. against the dam, and the flow of the spring increased to 0.52 sec-ft., after which it dropped back to 0.10 sec-ft., or slightly less than its original flow. When the water in the reservoir reached Elevation 2 260, or 22 ft. against the dam, during the 1910 season, new springs made their appearance near the foot of the spillway, away from the dam, the water apparently coming along a line of juncture between limestone and granite. The flow gradually increased to 0.99 sec-ft., with the reservoir water at 2 271.5, and ceased after the reservoir water had dropped back to Elevation 2 260. About one-half of the above flow came from a single point.

It should be stated, in this connection, that the excavation for the spillway revealed a rather unsatisfactory rock, with seams and lines of juncture between various formations, passing clear through the ridge. The excavated material broke up quite fine, and was dumped against the reservoir slope of the ridge, but was not rolled or compacted in any mechanical way. It is not improbable that this loose dump will gradually compact, due to the action of the water, and may act to some extent as a blanket.

The springs involve no special loss of water, as the water in the reservoir will be low until the commencement of the flood season, which coincides with the commencement of the irrigation season. The water comes through entirely clear, and the leakage constitutes no element of danger to the dam or to the spillway.

The flow from Scotch Creek, at the eastern base of the gravel bar to the east of the reservoir, has remained stationary, flowing at the rate of 0.75 sec-ft. at the point where measured, a mile below the upper springs, except that it varied slightly and rose to 0.86 sec-ft. after heavy rains, and independent of the elevation of water in the reservoir.

Some water was flowing near the toe of the dam during the process of hydraulicking. Since the work stopped the flow has been measured and has fallen from 0.61 sec-ft. on July 30th, to nothing on September 10th, the reservoir in the meantime being emptied. To what extent this flow indicates leakage through or under the dam, or may be attributable to a draining out of the dam itself, is as yet uncertain.

The dam may be considered as a loose rock and gravel structure with a puddle core. Some stratification may have occurred before the artificial core was started, which would undoubtedly lead to some seepage. It is also possible that the sheet-piling may not make an entirely satisfactory connection with the side-hill rock, although the piles were driven to contact with the rock as hard as was feasible.

The construction of the dam consumed more time than was expected, due largely to a smaller carrying capacity of the hydraulicking water than had been estimated, resulting from the large percentage of rock in the pits. The aggregate contents of the pits were barely sufficient for completing the dam, and left no margin of choice in materials.

The design of the dam was approved by the Director and the Chief Engineer of the Reclamation Service, and also by Mr. A. J. Wiley, as Consulting Engineer. The designs were made and construction work was commenced under the general supervision of Mr. E. G. Hopson, as Associate Supervising Engineer, and the writer, and was completed under the supervision of Mr. C. H. Swigart. The project was in immediate charge first of C. Andersen, M. Am. Soc. C. E., and later of Mr. Ferd Bonstedt, and the work was in personal charge of Mr. Lars Bergsvik, with O. Laurgaard, M. Am. Soc. C. E., as Engineer.

Other Earth Dams of the United States Reclamation Service.

It may be interesting to make comparison with sections of other earth dams built or being built by the Reclamation Service. Fig. 8 shows cross-sections of the following dams:

Clear Lake	Dam,	Klamath	Project,	Oregon-California.
Bumping Lake	"	Yakima	"	Washington.
Snake River	"	Minidoka	"	Idaho.
Deer Flat	"	Boise	"	Idaho.
Belle Fourche	"	Belle Fourche	"	South Dakota.
Avalon	"	Carlsbad	"	New Mexico.
Pathfinder Dike,		North Platte	"	Wyoming.

It will be noted that every cross-section shows open drainage material on the down-stream side, with the single exception of the Belle Fourche Dam, which is built entirely of clay, the nearest gravel being 5 or 6 miles distant.

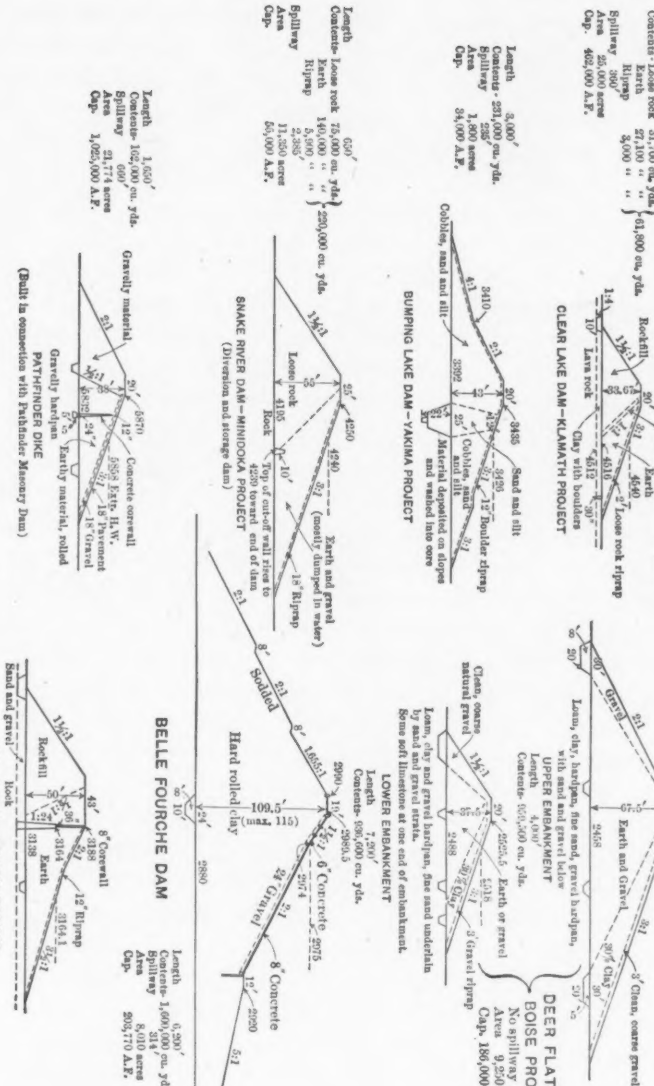
The absence of core-walls is also a general feature, barring the Pathfinder Dike and the Avalon Dam. In the former a core-wall was adopted because of the uncertainty as to the tightness of the available material, which had to be obtained from such scanty earth cover as could be found in the vicinity of the dam, or from the breaking up of the gravel hardpan, which is the only material of which large quantities were at hand.

The Avalon Dam, as a part of a private irrigation system, had twice failed. The first failure was due to over-topping. The cause of the second failure is not known. Parties having some familiarity with the conditions deny that the second failure was due to burrowing animals. The possible cause may have been the presence of a considerable quantity of soluble gypsum in the earthy material in the dam, which in canal banks has caused much trouble, and the menace of which is removed by the core-wall.

A water slope of 3 : 1 predominates, the Belle Fourche Dam being the most notable exception, the slope in that case being 2 : 1 and being faced with 8-in. concrete slabs laid on a bed of gravel.

Most of the dams mentioned above have been built by the ordinary spreading, sprinkling, and rolling method, which for briefness may be referred to as the mechanical, as compared with the hydraulic method.

In the Minidoka Dam, the earth facing of the loose rock was mechanically moved and dumped. It was dropped in water, and spreading and rolling were thereby obviated. The material was not subject to the sorting action of the water, and remained mixed as it came from the pits.



CROSS-SECTIONS OF
U.S. R.S. EARTH DAMS
FIG. 8.

In the Bumping Lake Dam, the material was moved mechanically to the slopes, mostly by dump-cars, and was washed by water-jets supplied from hose-nozzles. The large cobbles, and considerable gravel remained in place, while the sand and silt moved into the center, the coarse sand settling on the inner slopes and the fine sand and silt in the central pond.

Hydraulic Versus Mechanical Method of Dam Construction.

The hydraulic process has been used during the last few years on numerous dams, especially in the West. It appears to have decided advantages where circumstances render it applicable, as has been fully pointed out by J. D. Schuyler,* M. Am. Soc. C. E. In observing hydraulic work of this character, the writer has become strongly impressed with certain phases of this subject, which it may be opportune to submit for consideration.

Design.—In designing a dam to be built dry, it is constructively permissible to place the water-tight element in any position in the dam, where deemed best, to build it of any one of the various materials available, or any mixture of them, and to fix for it any dimensions desired.

No such freedom of choice exists with a dam to be built by hydraulicking. As to the position of the core in the dam, construction will be greatly facilitated by placing it between two dumps, and, if intended to be built of material in which clay predominates, stability may make this position practically necessary.

The core material may be obtained from a special pit, or may be washed into a pond from the slopes. In the latter case the core dimensions cannot be fixed arbitrarily, but must result from the proportionate contents of pit material, which it is difficult, and may be impossible, to ascertain definitely in advance.

If it be deemed desirable that the material in the core shall be thoroughly consolidated by drainage and pressure by the time the work is completed, another difficult problem enters into the design, as clays and silts vary greatly in regard to their retentiveness of surplus water. Little definite information is at present available on this important point.

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* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 196.

Materials in Pits.—It is a rare occurrence that the material in extensive borrow-pits runs uniform. When changes are encountered during excavation the mechanical method of construction permits a ready adjustment by opening other pits, independent of distance and elevation, other than as affecting cost.

The hydraulic method, while more dependent on suitable proportions in pit material, is less elastic in adapting itself to changing and unexpected conditions, as pits must be located above a certain inclined plane, in which dirt flumes can be built, unless at the expense of general reconstruction and the possible addition of pumping.

Variation of Materials in Pits.—When the material from pits is handled dry it is constantly under easy inspection, and the disposition of the various materials can be readily controlled. If at any time the more open materials, intended for the drainage part of the dam should preponderate, that part of the dam can be made to advance for a short time ahead of the remainder.

With the hydraulic process such adaptation is difficult, and at times not feasible, as the ponds must be maintained as slowly rising levels. At a certain stage of the work operations may be well adjusted to the momentary proportions as they are sluiced in, when an unusually coarse part of the pit may be reached, and a large excess of sand and gravel may be dumped before the necessity is realized of a more rapid change of discharge points. The sand slope will then build itself forward into the puddle pond, at first not visibly because it is below the surface of the muddy water, and, especially when the silt-core narrows up toward the top of the dam, any stratification which may occur may easily reach through the core. Again, an excess of clay or silt may be encountered in the pit, which, if sluiced in for too great a length of time, may give rise to a layer in the dam, such as may produce local slides.

The writer has heard it argued that the silt-laden water flowing over sandy layers in the core will cause the silt to penetrate the sand, but he has never found this to be the case, unless the sand had been actually picked up by the current. Over and over again he has found clean layers of sand reaching out into the silt core without any apparent mixing, and the fact that the pond may at a certain stage be tight vertically, by no means proves that the material below is not subject to leakage horizontally. The above is in accord with the gen-

eral occurrence of pure sand strata in alluvial lake deposits, alternating with layers of silt.

Character of Core Material.—Masonry, clay, and earth mixtures have their peculiar advantages for use in cores, and each in turn may be the best to adopt in individual cases. When clay is used, it is generally the object to put it in place in a tough, thoroughly puddled, and compact condition, and to give it a position in the dam, where it will be well protected from atmospheric influences. Earth mixtures aim at obtaining tightness by the use of coarser materials, such as broken rock, or sand and gravel, mixed with sufficient clay or silt to fill the interstices, thereby securing at the same time stability and freedom from shrinkage in case of drying.

Experience with the hydraulic process is not sufficiently advanced to define with certainty the thickness of a clay core which, if hydraulicked into place, can be depended on to become thoroughly compacted in a short time by pressure and drainage. If clay is sluiced into a pond it arrives in place in a super-saturated condition, and the surplus water will only be given off slowly. If the mass be great, the interior may remain in a semi-liquid condition for a long time, endangering the stability of the entire dam.

The ultimate removal of surplus water must be accompanied by reduction of the space occupied by the clay. This implies not only downward settlement, which, if excessive, is in itself objectionable, but also horizontal contraction, causing vertical cracks, which is highly undesirable and may become dangerous.

The writer is led to believe from his experience that a tough consolidated core of large dimensions, in which the clay occupies a minimum of space, cannot be produced with the hydraulic process as usually practised. If such results have been obtained in cases which have not come to his attention, he believes that a statement of the conditions and of any special means of drainage, which may have been used, would be highly interesting.

As regards a core of mixed materials in reasonably controlled proportions, it is doubtful whether such a core can be made by the hydraulic process; for the one strongly marked feature of handling mixed materials with water is that immediately on the formation of slopes active segregation takes place.

Probably the nearest approach to a perfect core of great thickness

which can be hydraulicked, is one composed of fine sandy silt, such as is generally found in the arid West, having little cohesiveness, good self-drainage qualities, becoming hard and solid after a short time, and yet being, if not perfectly, at least practically water-tight.

If the core be built of whatever fine material is washed into a central pond from the slopes, there is danger of stratification, as previously stated. Opportunity for visible inspection, such as is continuously afforded with the mechanical process, exists only at infrequent intervals, when ponds are drained, and after large masses of material have been washed in. Therefore, dependence must be placed on sounding, which is very uncertain in revealing actual conditions.

In order to counteract stratification, mechanical mixing has been attempted, such as forcing down and twisting paddles, having men walk through the pond with rubber boots, etc. At best these methods are doubtful in their results. Moreover, they are likely to be entirely ineffective if stratification has proceeded beyond a certain point, for even thin layers of coarse sand offer considerable resistance to paddling.

Limiting Height.—Some of the foregoing considerations suggest that with clayey core materials, the core thickness which may be needed at the base of a high dam may be greater than can be placed with safety by the hydraulic method, by reason of uncertainty of drainage and proper consolidation. The use of such material, in connection with hydraulicking, would place a practical limit on the height to which a dam of this type can be safely built.

With the same material, placed mechanically, and without excess of water, there is no constructive limit to the feasible core thickness, and the height to which an earth dam can be safely built, as far as this consideration is concerned.

Masonry Core.—If a masonry core is adopted as part of the design, its construction interferes but little with the work, when carried on dry. In case of hydraulicking, however, the wall divides the pond into two parts, which are likely to stand at different levels and to subject the wall to undue pressures during construction. In one instance, the writer observed a large hole cut through a core-wall in a dam being built by the hydraulic process, apparently for the sole purpose of equalizing pressures.

A core-wall prevents the up-stream part of the pond from draining in a down-stream direction. Should circumstances render it

desirable to commence reservoir storage prior to completion, drainage requirements compel the maintenance of a much greater difference in level between the surface of the reservoir water and the top of the completed work than might otherwise be safe and permissible with mechanical construction; and the same holds true if the dam be given an ordinary clay or silt core.

Drainage Element.—Considering rock-fill dams as a class by themselves, there are few of the older earth dams which possess provision for drainage to the extent of having their down-stream sections consist of open material. Such provision has the disadvantage of lessening the amount of tight material which resists the flow of water through the dam, and may thus lead to increased percolation, but it has the obvious advantage of preventing the line of saturation through the dam from intersecting the down-stream slope above the lower toe. All tendency to slough is thus avoided, which may in most cases be regarded as of far more importance than a slight increase in percolation.

The hydraulic process, applied to mixed materials, is admirably adapted through segregation on slopes, to securing such drainage element in the down-stream portion of the dam, although a similar section on the up-stream side of the dam, if it should have dimensions greater than is necessary for core protection, may well be considered objectionable, and cannot always be avoided. The above advantage becomes of little consequence where, as was the case at the Cold Springs Dam, a good drainage material is available for placing economically by the mechanical method.

Time.—The comparative amount of time required for construction must depend on local conditions, and each system may have the advantage where conditions favor it. What is almost of equal importance to the total time needed, and may be of more importance when building across streams subject to violent floods, is the ability to estimate the required time closely in advance. Where mechanical equipment is used, the factors affecting time are generally under close control. With the hydraulic method, the output is dependent on continuous water supply, percentage of solids carried, and freedom from interruption through breaking of water supply or clogging of dirt flumes. Each of these factors is likely to contain elements of serious uncertainty, to which may be attributed the apparent fact that

many hydraulic-fill dams have required far more time for their completion than had been originally estimated.

Cost.—Under favorable conditions, of sandy silt in high banks, a continuous and abundant water supply under a satisfactory pressure, and well-adjusted flume grades, the quantity of material that can be hydraulicked in a short time is remarkable, the proportion of water-carried material sometimes exceeding 25 per cent. In such cases the cost per cubic yard may be surprisingly low.

With coarse material the percentage carried by water is rapidly cut down, even with increased flume grades the time lost by clogs may be relatively great, so that the actual performance may become small and the labor cost high.

Even with small labor cost, the plant charges of a hydraulic installation, as compared with mechanical equipment, may be so high as to throw the balance in favor of the dry method of construction.

Comparing the actual examples of construction work described in this paper, the cost per cubic yard of material in the Cold Springs Dam averaged 43 cents and in the Conconully Dam 51 cents, both based on bank measurement, while the relative plant charges, on the same basis of measurement, were approximately 12 and 20 cents, making the balance of the cost practically alike.

Such comparisons are decidedly dangerous, however, and the above should not be looked on as an argument against possible cheapness of the hydraulic process. The mere cost figures do not show in themselves the difference in conditions, and, while it is true that at Cold Springs the material had to be raised and carried farther, the costs at Conconully were greatly affected by the remoteness of the location, the necessity of stopping work over winter, and materially smaller quantities. There can be little question that the construction at Conconully under the existing conditions would have been higher in cost if any other than the hydraulic method had been used.

The difficulties, above pointed out, in estimating the required time for hydraulic construction apply equally to cost estimates, in which respect greater certainty can probably be had when the mechanical process is adopted. This condition is reflected by the usual inability of obtaining unit contract bids on hydraulic work, although this is undoubtedly due in part to the lack of contractors who have even a limited experience in this line of construction.

Conclusion.—The foregoing comparison has been made in some detail to indicate what are believed to be the principal factors, which should receive attention, when the application of the hydraulic method to the construction of large earth dams is under consideration.

It appears to the writer that the hydraulic process has important advantages in special cases, and also that it has decided limitations, not at present well defined. Its potential economical possibilities may at times be utilized to their full extent, and it may under exceptional conditions accomplish results not otherwise readily obtainable.

The mechanical process has the important advantage in the construction of high earth dams, designed to contain great masses of clay or material in which clay largely predominates, of permitting immediate consolidation and avoiding the dangers from instability and possible future excessive settlement and shrinkage, which may result from initial surplus water, such as would be necessarily introduced by the application of the hydraulic process.

In cases of doubt, the hydraulic and mechanical methods being equally applicable and showing little difference in cost, it is believed that the mechanical method deserves the preference, by reason of greater certainty of results as to quality, time, and cost.

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PAPERS AND DISCUSSIONS

STEEL CENTERING USED IN THE CONSTRUCTION OF THE ROCKY RIVER BRIDGE, CLEVELAND, OHIO.

By Wilbur J. Watson, M. Am. Soc. C. E.

To be Presented June 7th, 1911.

There has recently been completed, near Cleveland, Ohio, a concrete arch bridge having a clear main span of 280 ft. and a rise of 80 ft. The length of the structure over all is 708 ft. The roadway is 40 ft. wide, and is flanked by two 8-ft. footwalks. The structure was built by Cuyahoga County, over Rocky River, on Detroit Avenue, and connects the suburban villages of Lakewood and Rocky River. It will carry a double-track interurban railroad, and heavy vehicular traffic.

The general design is quite similar to that of the Walnut Lane Bridge, at Philadelphia, but a radical departure from the usual methods was made in the construction, as steel, instead of timber centering, was used for the main arch.

The contract for this bridge was let on August 22d, 1908, and prior to that time, a contracting firm, desiring to submit a bid, retained

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

the writer to design and estimate the cost of the centering and forms required.

The writer made preliminary plans for both timber and steel centers, including several types of the latter, and finally decided on the use of the three-hinged arch as being the most practicable and economical. Among the advantages which the steel centers possessed were the following:

1st.—Economy: These centers were estimated to be at least $15 000 cheaper than timber centering of the usual type.

2d.—Freedom from danger of being carried out by ice. It was evident that these centers would have to be in the river at least one winter, and there was great danger of losing them if timber centering of the ordinary type were used.

3d.—The three-hinged arch is a statically determinate structure, and it was a simple matter to compute the exact stresses and deflections for each position of the load.

4th.—The deflection of the hinged arch under load was found to be far less than that of centers of the truss or beam type.

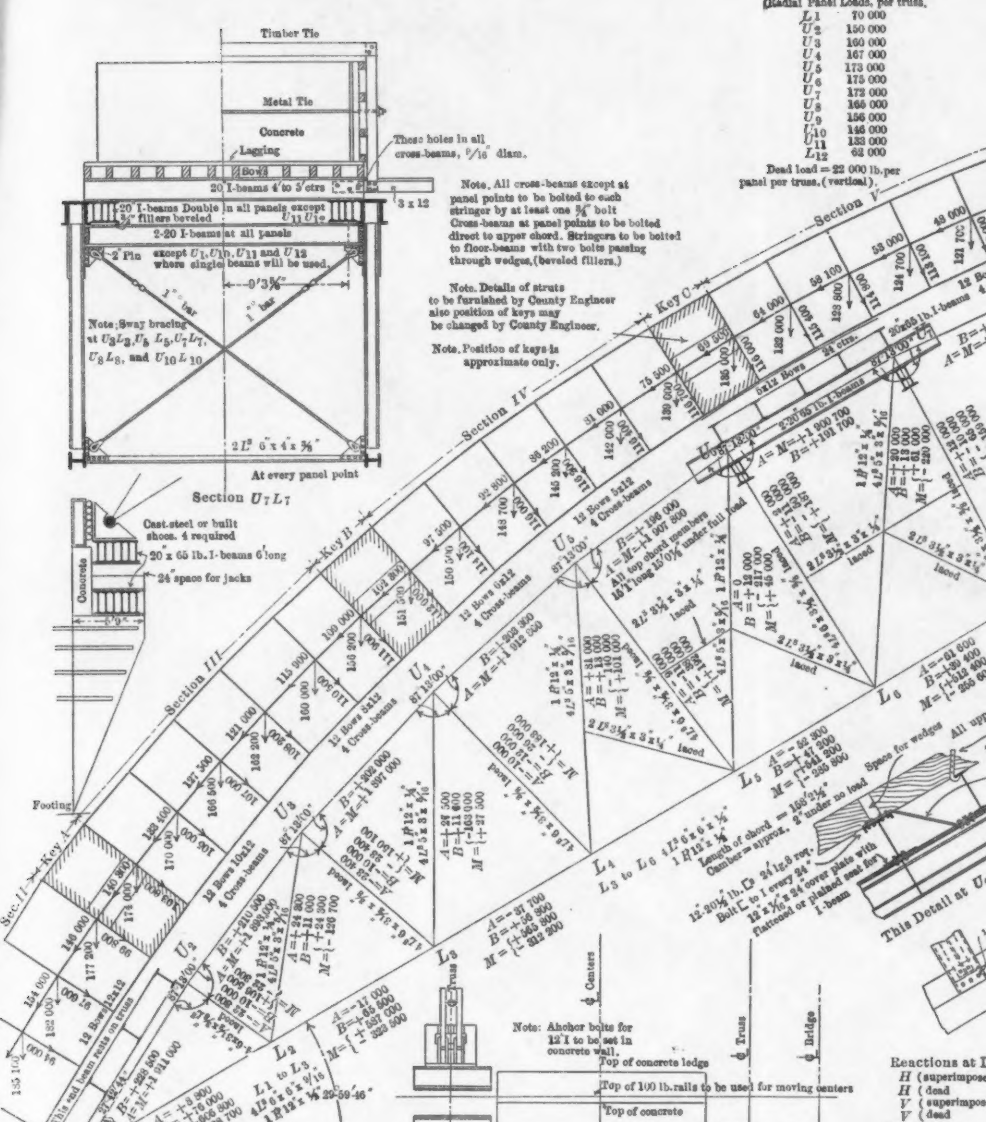
As the use of steel centers for the construction of permanent masonry arches of such long span had never been tried before, the latter was an essential consideration. As far as the writer is aware, these were the first steel centers to be used for any but very short spans.

The steel centers used in the construction of the 150-ft. arches of the Delaware River Bridge of the Delaware, Lackawanna and Western Railway were designed some months later, when the centers described herein were well under way in the shop.

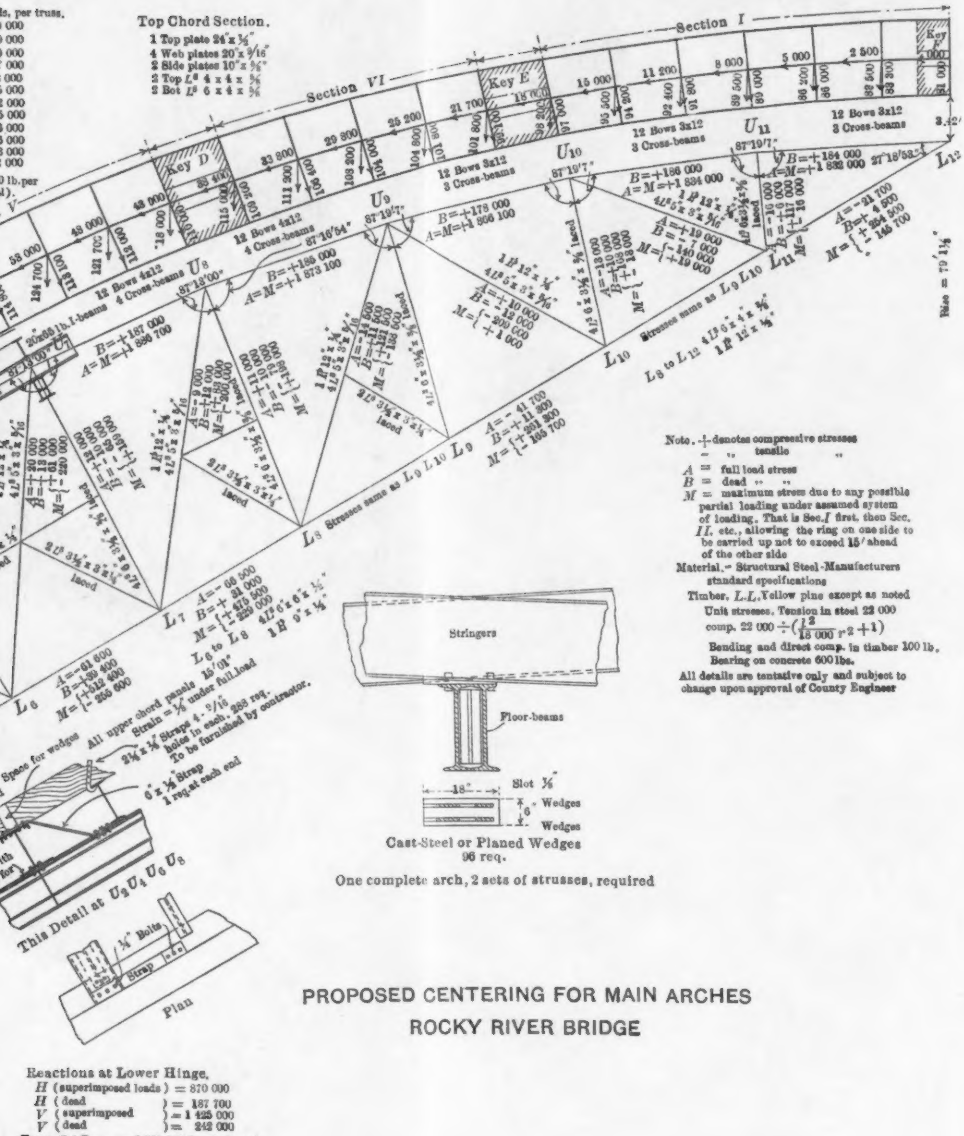
After the contract was awarded, the writer was instructed to prepare a complete design for these centers.

In computing the stresses, it was assumed that only the radial component of the weight of each voussoir was to be carried by the steel centers, and that the tangential component would be carried by the timber bows, or by reinforced concrete struts placed in the key sections.

In carrying out the work, both of these provisions were made, the bows being of sufficient strength to carry the entire tangential compo-



Radial Panel Loads, per truss.
L_1 70 000
U_2 150 000
U_3 160 000
U_4 167 000
U_5 173 000
U_6 175 000
U_7 172 000
U_8 165 000
U_9 156 000
U_{10} 146 000
U_{11} 133 000
L_{12} 62 000
Dead load = 22 000 lb. per panel per truss. (vertical).
Timber Tie
Metal Tie
Concrete
Lagging
Bows
These holes in all cross-beams, 9/16" diam.
20 I-beams 4' to 5' ctrs
3 x 12
20 I-beams Double in all panels except $U_{11} U_{12}$
3/4" fillers beveled
2-20 I-beams at all panels except U_1, U_{10}, U_{11} and U_{12} where single beams will be used.
2" Pin
1" bar
Note; Sway bracing at $U_3L_3, U_5L_5, U_7L_7, U_8L_8$, and $U_{10}L_{10}$
2 L^s 6" x 4" x 3/8"
At every panel point
Section U_7L_7
Note. All cross-beams except at panel points to be bolted to each stringer by at least one 3/4" bolt. Cross-beams at panel points to be bolted to upper chord. Stringers to be bolted to floor-beams with two bolts passing through wedges. (beveled fillers.)
Note. Details of struts to be furnished by County Engineer also position of keys may be changed by County Engineer.
Note. Position of keys is approximate only.
Cast-steel or built shoes. 4 required
20" x 65 lb. I-beams 6' long
24" space for jacks
Concrete
Footing
Sec. II
Key A
Section III
Key B
Section IV
Key C
Section V
12 Bows 12x12
12 Bows 10x12
12 Bows 8x12
12 Bows 6x12
12 Bows 5x12
4 Cross-beams
5x12 Bows 24 ctrs.
This end beam rests on truss
U_2 U_3 U_4 U_5 U_6 U_7
L_1 L_2 L_3 L_4 L_5 L_6
87°13'00"
All top chord members 15" I long 15'0 1/2" under full load
L_1 to L_3 4 L^s 6" x 6" x 9/16" 1 Pl 12" x 1/2"
L_3 to L_6 4 L^s 6" x 6" x 1/2" 1 Pl 12" x 1/2"
Length of chord = 156' 2 1/4"
Camber = approx. 2" under no load
Space for wedges
12-20 1/2 lb. [s 24' lg. 8 req.
Bolt [to I every 24"
12" x 1/16" x 24 cover plate with flattened or planed seat for I-beam
This Detail at U
Springing Line.
1/2 Span = 137.0236'
29°59'46"
29°30'30" to rad. of curve of upper chord.
2.98
1.68
Note: Anchor bolts for 12" I to be set in concrete wall.
Top of concrete ledge
Top of 100 lb. rails to be used for moving centers
Top of concrete
Half Elevation of Abutment
℄ Centers
℄ Truss
℄ Bridge
3'0"
11'7"
17'0"
Reactions at
H (superimposed
H (dead
V (superimposed
V (dead
Tangential Force =



PROPOSED CENTERING FOR MAIN ARCHES
ROCKY RIVER BRIDGE

nents of the loads, and reinforced concrete struts being also provided, as indicated on the stress diagram, Plate XL.

This assumption has been criticized, and the best proof of its accuracy which the writer can offer is the fact that the observed and computed deflections corresponded precisely, and they would not have done so had not this assumption as to the distribution of the load been correct. The bows were simply laid on the cross-beams, being left free to move longitudinally thereon, and were carefully framed with butt joints, so as to act as arch ribs.

The design was computed on the assumption that the arch ring would be concreted in 15-ft. transverse sections, or voussoirs, and it was assumed that a section on one side of the arch might be completed in advance of the corresponding section on the other side. It was also assumed that the placing of the voussoirs might take place in any order, symmetrically about the center. Before checking, however, the order of loading had been determined, and the computations were based on the assumption of such order of loading.

A unit stress of 21 000 lb. per sq. in. on medium open-hearth steel was used, making the proper reduction for columns according to Gordon's formula for pin-connected columns.

It was decided to use bolts instead of rivets for all field connections, in order to facilitate removal, after the completion of the work.

These bolts were turned to a diameter of $\frac{29}{32}$ in., and placed in $\frac{15}{16}$-in. reamed holes. All holes for field connections were reamed in the field, using pneumatic reamers.

The centers were designed to be erected in an approximately vertical position, and then lowered into position from the banks. The contractor, however, preferred to erect them on light timber false-work, which was to be removed as soon as the trusses were in place. An 800-ft. cableway, installed for handling the concrete, was also utilized in erecting the centers, and in their removal.

The weight of steel in these centers was approximately 400 tons, of which about 160 tons were steel I-beams which were used in the completed structure, being placed across the opening between the two arch ribs, transversely to the axis of the bridge, and encased in concrete.

The use of so large a part of the steel in the permanent structure

was quite an item contributing to the remarkable economy of these centers. The cost was approximately as follows:

Material and fabrication..............	$20 000	
Freight and erection..................	5 500	
Dismantling	1 200	
Total		$26 700
Value of steel I-beams used in permanent structure....................	$7 000	
Value of remainder of structure as scrap material	2 500	
		9 500
Net cost of centers.....................		$17 200

The above figures, exclusive of wedges, take no account of overhead or engineering charges, nor the cost of timber lagging and moving the centers, the latter being a very small item.

The two most troublesome problems to be solved were the provision for motion of the steel arches, due to temperature changes, and the design of the lowering jacks.

The County Engineer's specifications provided that the timber centering contemplated therein should be kept wet during the construction of the keys, and the writer's original intention was to keep the steel centers wet during the concreting and setting up of the last key (at the crown). As city water would be used, and at a practically constant temperature as it emerged from the pipes, it would seem that by this means, and by using canvas to screen the steel from the direct rays of the sun, the position of the crown hinge might be kept at a constant elevation for the length of time necessary for the concrete in the key to harden.

It was decided, however, that this was unnecessary, as the arch was not reinforced, and any rise of the center hinge would only serve to open up the voussoir joint at the crown.

This voussoir, therefore, was concreted on a cool day, and the surfaces of the two adjacent arch sections were oiled in order to prevent an immediate bond, so that there would be no danger of the key concrete being prematurely loaded; and the joint at the crown was allowed to open and close with temperature changes. The opening was as great as $\frac{5}{16}$ in. at one time. The rise and fall of the center hinge was so sensitive to changes of temperature, that Mr. Stevens, the

FIG. 1.—STEEL CENTERS FOR ROCKY RIVER BRIDGE.

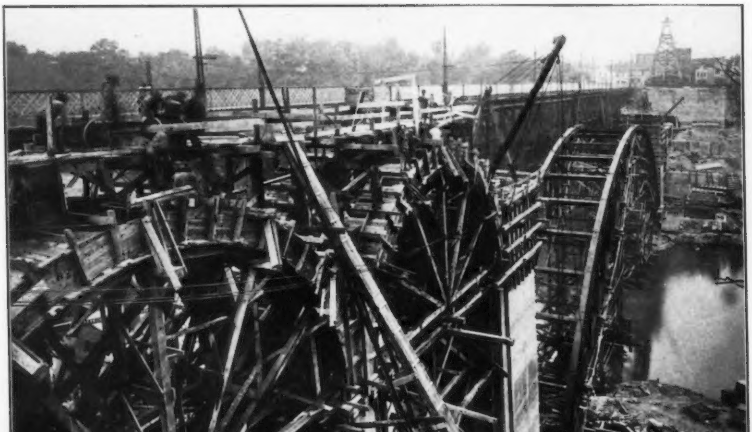


FIG. 2.—APPROACH SPANS, AND STEEL CENTERS, ROCKY RIVER BRIDGE.

Resident Engineer, states that he could determine the temperature of the air by measuring the opening.

The computed deflection of each truss, or half arch, under full load was 2 in., and the trusses were cambered this amount. The observed deflection corresponded with this precisely. It is evident that, in the case of a reinforced arch, it would have been necessary to carry out the original intention of wetting down the centers, but it is difficult to see how the principle of reinforcing an arch of this kind can be defended, because the live load is practically negligible in comparison with the dead load, and the curve of the arch can be adapted to the curve of the equilibrium polygon, thus practically eliminating all bending and shearing forces in the arch ring.

Plate XL shows the stress sheet for one-half of one center, and details of the bows, lagging, etc. It will be noted that the arch ring is divided into voussoirs, which are given Roman numerals corresponding to the sequence of concreting. The keys are designated by letters, also corresponding to the sequence in which they were concreted.

It was found by experiment that a concrete composed of one part Portland cement, one part sand, and two parts stone would attain as great a strength in 7 days as the concrete used in the arch ring would attain in 30 days. The rich mix, therefore, was used in the keys, in order that the centers might be the sooner removed. Table 1 gives the results of these tests on concrete.

TABLE 1.—CRUSHING STRENGTH OF SIX-INCH CONCRETE CUBES; ROCKY RIVER BRIDGE.

Kind of concrete.	Age.	Crushed.	Cracked.	Kind of concrete.	Age.	Crushed.	Cracked.
1 : 3 : 5	30 days.	1 718	1 363	1 : 3 : 5	6 months.	2 781	2 509
"	" "	2 045	1 527	"	" "	2 727	2 454
"	" "	2 018	1 418	"	" "	2 127	1 800
"	" "	1 936	1 636	"	" "	1 909	1 527
"	" "	1 909	1 527				
"	" "	2 454	2 072	1 : 2 : 4	30 days.	2 781	2 127
"	" "	2 182	2 018	"	" "	2 836	2 836
"	" "	2 509	2 454	"	" "	3 270	2 181
"	" "	2 073	1 265	"	" "	3 380	770
"	" "	2 509	1 582	"	" "	3 272	1 910
"	" "	2 564	1 582				
"	" "	1 854	1 091	"	90 "	2 672	2 180
				"	" "	2 836	1 910
"	90 "	2 563	2 127	1 : 1 : 2	7 "	2 945	1 091
"	" "	2 509	1 964	"	" "	3 436	2 727
"	" "	2 127	1 361	GRANOLITHIC FACING.			
"	" "	1 936	1 909				
"	" "	2 891	1 909	1 : 1 : 2	90 days.	2 727	2 727
"	" "	2 836	1 963	"	" "	2 945	2 836

Tables 2 and 3 give the observed deflections of the steel arch under different conditions of loading.

TABLE 2.—ELEVATIONS ON ALTERNATE POINTS OF EACH HALF ARCH TAKEN DURING PLACING OF CONCRETE; ROCKY RIVER BRIDGE.

EAST SIDE.

Date.	Ct. Pin.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	Temp.	Remarks.
8/ 6/09	76.819	74.540	66.150	52.315	33.495	10.415	84°	25 tons in place.
8/ 9/09	76.708	74.470	66.150	52.330	33.505	10.410	85°	465 tons in place, Sec. I.
8/11/09	76.683	74.458	66.123	52.368	33.508	10.408	74°	Sec. I and 6 ft. of Sec. II in place.
8/13/09	76.680	74.468	66.113	52.313	33.508	10.403	80°	Sec. II in place.
8/17/09	76.682	74.466	66.106	52.296	33.486	10.391	77°	¾ Sec. III in place.
8/19/09	76.699	74.544	66.109	52.294	33.464	10.384	78°	Sec. III in place.
8/24/09	76.762	74.482	66.102	52.262	33.422	10.367	86°	¾ Sec. IV in place.
8/26/09	76.782	74.493	66.103	52.233	33.408	10.368	85°	Sec. IV in place.
8/28/09	76.773	74.463	66.038	52.188	33.388	10.358	73°	Sec. V and ½ VI in place.
8/31/09	76.777	74.426	66.006	52.166	33.381	10.351	80°	Voussoirs in place.

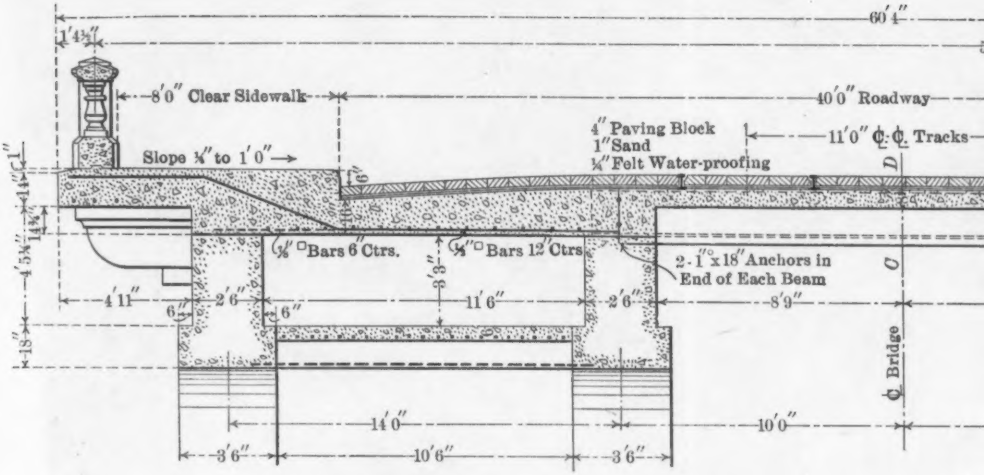
WEST SIDE.

Date.	Ct. Pin.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	Temp.	Remarks.
8/ 6/09	......	74.533	66.128	52.328	33.483	10.393	84°	25 tons in place.
8/ 9/09	......	74.455	66.210	52.310	33.490	10.400	85°	465 tons in place, Sec. I.
8/11/09	......	74.428	66.098	52.288	33.478	10.390	74°	Sec. I and about 6 ft. of Sec. II in place.
8/13/09	......	74.444	66.181	52.301	33.491	10.396	80°	Sec. II in place.
8/17/09	......	74.429	66.169	52.289	33.469	10.381	77°	¾ Sec. III in place.
8/19/09	......	74.461	66.161	52.281	33.456	10.376	78°	Sec. III in place.
8/24/09	......	74.463	66.078	52.243	33.408	10.363	86°	¾ Sec. IV in place.
8/26/09	......	74.453	66.058	52.213	33.388	10.353	85°	Sec. IV finished.
8/28/09	......	74.408	65.988	52.178	33.378	10.348	73°	Sec. V and ½ Sec. VI in place.
8/31/09	......	74.391	66.011	52.171	33.371	10.346	80°	Voussoirs in place.

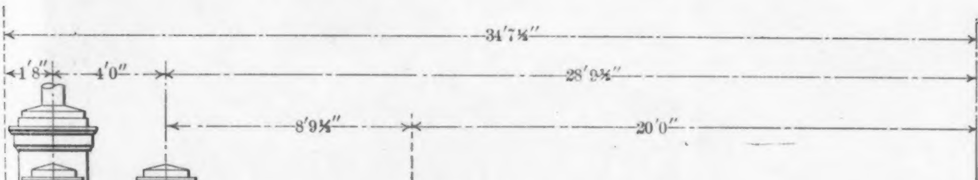
The rise and fall of the centers due to changes of temperature were very carefully noted, both before and after loading, as were also the deflections due to partial loadings.

The first concrete in the main arch rib was placed on August 6th, 1909, and the voussoirs were completed August 30th. The keys were run on September 3d to 9th, and the centers were struck on September 28th. When the centers were moved into position for the second rib, the first arch rib and the columns, up to the springing line of the spandrel arches, were in place.

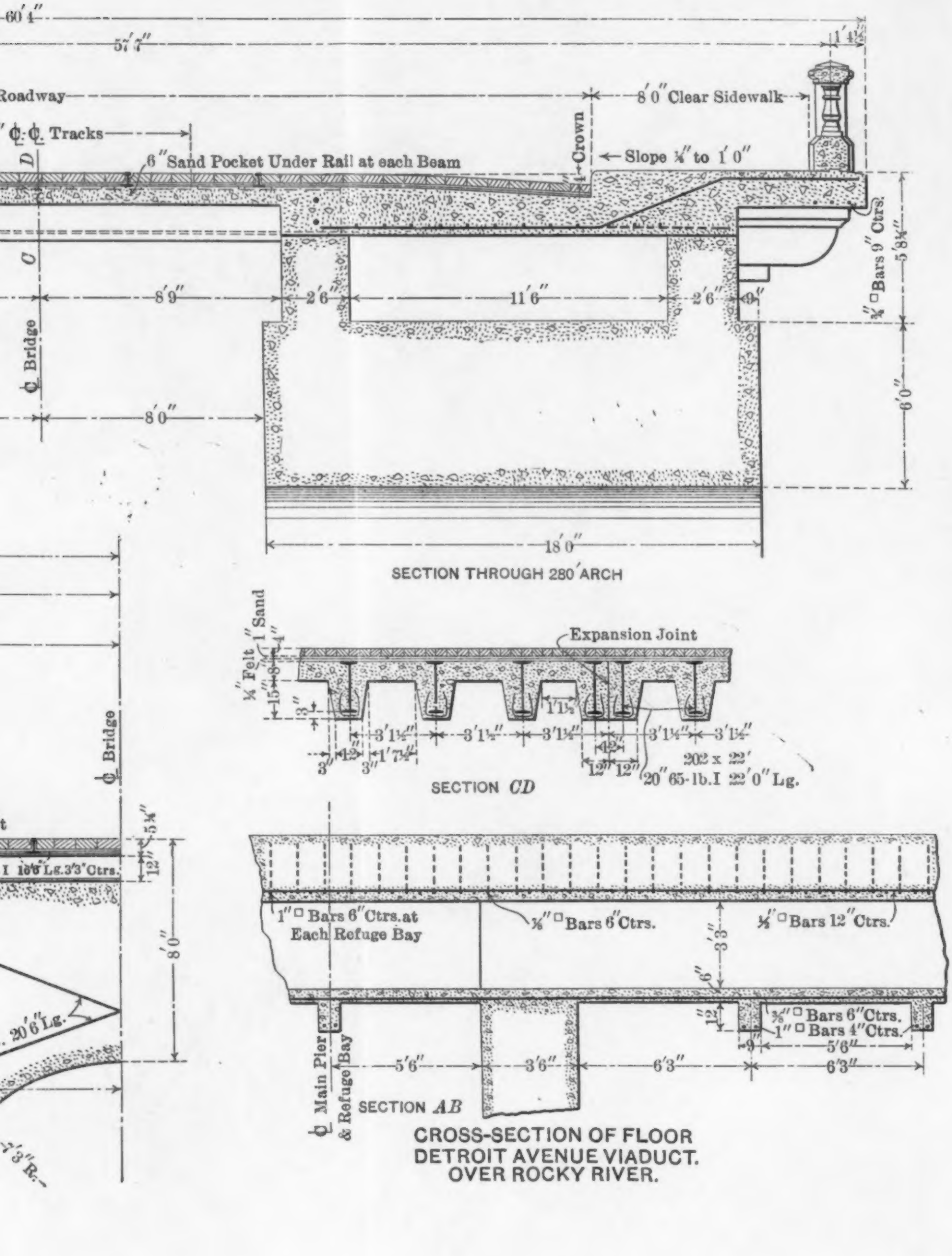
The unit stress in the arch rib due to its own weight is about 245 lb. per sq. in. The settlement of the crown when the centers were removed amounted to 0.038 ft. The curve of the intrados, as constructed, varies from the computed curve less than ½ in. at the point of maximum variation.



SECTION THROUGH SPANDREL ARCH



SECTION THROUGH REFUGE BAY



SECTION THROUGH 280' ARCH

SECTION CD

SECTION AB

CROSS-SECTION OF FLOOR
DETROIT AVENUE VIADUCT.
OVER ROCKY RIVER.

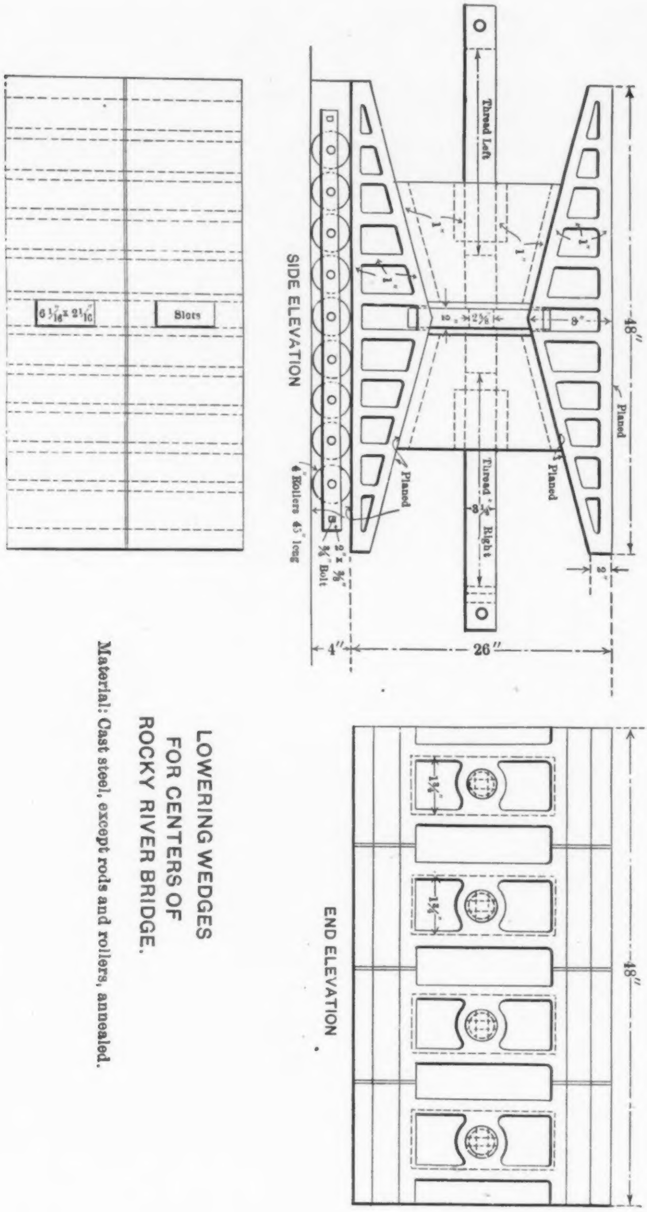


FIG. 1

LOWERING WEDGES
FOR CENTERS OF
ROCKY RIVER BRIDGE.

Material: Cast steel, except rods and rollers, annealed.

TABLE 3.—ELEVATIONS OF SOUTH CENTER PIN DUE TO CHANGES OF TEMPERATURE AFTER CONCRETE WAS PLACED; ROCKY RIVER BRIDGE.

Date.	Hour.	Elev.	TEMPERATURE.		Remarks.
			In shade.	In sun.	
9/ 9/09	1.00 P. M.	76.771	75°	....	Cloudy.
9/ 9/09	5.35 P. M.	76.758	68°	....	Raining. Key finished at 7 P. M. Temp., 65°.
9/10/09	8.30 A. M.	76.743	69°	....	Trans. reading same as 7.00 last night when key finished.
9/10/09	12.30 P. M.	76.762	74°	78°	Trans. reading same as last night and this A. M. Sun since 10.30.
9/10/09	3.00 P. M.	76.761	75°	....	Clear. Trans. reading same as last night and when key completed.
9/11/09	8.45 A. M.	76.754	69°	81°	Clear. Trans. reading same as last night and when key completed.
9/11/09	11.30 A. M.	76.754	71°	78°	Clear. Trans. reading same as last night and when key completed.
9/13/09	8.40 A. M.	76.770	79°	86°	Clear. Joint open W. side of key, $\frac{1}{16}$ in.
9/13/09	1.00 P. M.	76.785	85°	96°	" " " $\frac{1}{4}$ in.
9/13/09	4.00 P. M.	76.783	85°	87°	" " " $\frac{5}{16}$ "
9/14/09	8.55 A. M.	76.770	84°	100°	" " " $\frac{3}{16}$ "
9/14/09	1.45 P. M.	76.786	85°	93°	" " " $\frac{1}{4}$ "
9/14/09	4.10 P. M.	76.782	82°	85°	" " " $\frac{1}{4}$ "
9/15/09	9.00 A. M.	76.764	75°	78°	" " " $\frac{1}{16}$ "
9/15/09	12.45 P. M.	76.764	81°	....	Cloudy. " " $\frac{1}{8}$ "
9/15/09	4.00 P. M.	76.755	69°	....	" " " $\frac{1}{16}$ "
9/16/09	9.00 A. M.	76.749	69°	....	" " " $\frac{1}{32}$ "
9/16/09	1.00 P. M.	76.752	67°	78°	Joint open, $\frac{1}{32}$ in.
9/16/09	4.00 P. M.	76.754	67°	75°	" " $\frac{1}{32}$ "
9/17/09	9.00 A. M.	76.739	69°	78°	" " 0 "
9/17/09	1.05 P. M.	76.759	71°	82°	" " 0 "
9/17/09	4.00 P. M.	76.758	71°	80°	" " $\frac{1}{16}$ "
9/18/09	8.55 A. M.	76.748	67°	85°	" " 0 "
9/18/09	11.50 A. M.	76.751	67°	83°	" " $\frac{1}{32}$ "
9/20/09	8.50 A. M.	76.750	77°	82°	" " $\frac{1}{16}$ "
9/20/09	1.00 P. M.	76.780	83°	86°	" " $\frac{1}{4}$ "
9/20/09	4.00 P. M.	76.780	80°	85°	" " $\frac{1}{8}$ "

It was at first intended to use hydraulic jacks for lowering the centers, and the writer, in connection with the Duff Manufacturing Company, of Pittsburg, worked out a plan for lowering, using 32 jacks, in nests of four, each jack having a lowering capacity of 105 tons; nests of eight were to be operated by a single pump and reservoir.

As it would be necessary to lock these jacks, owing to the length of time that they would be called on to sustain their load in one position, the Duff Manufacturing Company, designed a special locking device which could be released without lifting the ram.

In addition to the hydraulic jacks, the use of sand-boxes, screw-wedges, and screw-jacks was investigated. It was found that the sand-boxes were a little cheaper than any of the other devices, but, owing to the fact that these boxes would be immersed in water (as their position was below mean water level), it was thought that it would be practically impossible to keep the sand dry. The sand-boxes, as contemplated, were to be of cast steel, and the estimated cost was but little

FIG. 1.—CONCRETING ARCH RING, ROCKY RIVER BRIDGE.

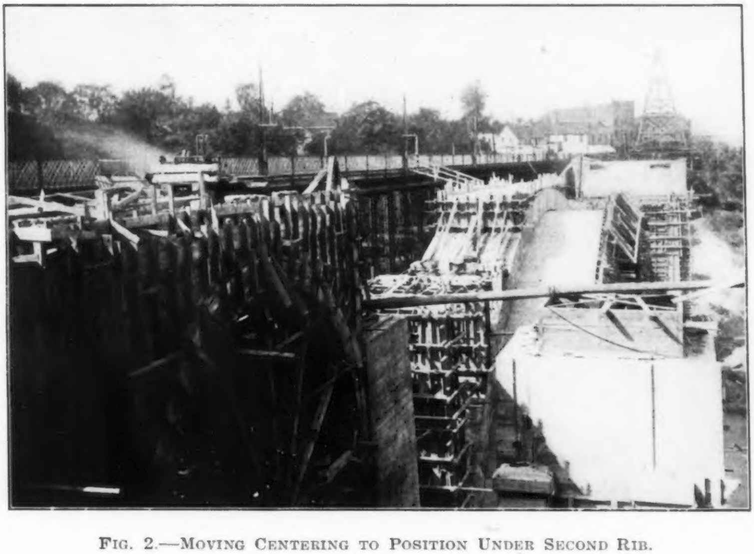


FIG. 2.—MOVING CENTERING TO POSITION UNDER SECOND RIB.

CENTERING, AND CONCRETE ARCH, ROCKY RIVER BRIDGE.

less than that of the double-acting screw-wedges, which were finally adopted.

The design of these double-acting steel wedges, as finally used, Fig. 1, was suggested by A. M. Felgate, County Bridge Engineer, while looking at the writer's design for a similar single-acting screw-wedge, and appeared to be exactly what was needed. They were made of carefully annealed cast steel, in four sections, planed on all sliding and bearing surfaces, and very carefully adjusted to uniform height when set in position. The motion allowed for was a vertical drop of $6\frac{1}{2}$ in., and the slopes were calculated so that locking would not be required to hold them under full load. They were lowered by four men on each screw. Each wedge weighed approximately 1 350 lb.

Next in cost to the steel wedges were the screw-jacks, each of which was designed to lower 105 tons and lift 22 tons. The hydraulic jacks were found to be the most expensive.

Estimated cost of sand-boxes....................	$1 300
" " " screw-wedges	1 650
" " " screw-jacks	2 000
" " " hydraulic jacks................	2 400

Particular attention is called to the high class of shop workmanship required on the centers. The reaming was done with unusual care. The planed joints were required to be perfect, and were the best the writer has ever seen. In fact, the workmanship was better than that required for standard railroad bridge work. In view of this, and also of the fact that the structure was free from impact, the writer is of the opinion that the unit stresses assumed were not unreasonable.

The bridge was built by Cuyahoga County, under the direction of A. B. Lea, former County Engineer, and F. R. Lander, present County Engineer. The structure was designed by A. M. Felgate, County Bridge Engineer. The Contractor was the Schillinger Brothers Company. The Resident Engineer was A. L. Stevens. The centers were built by the Interstate Engineering Company, and the wedges were cast by the Variety Iron Works Company, of Cleveland.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS
This Society is not responsible for any statement made or opinion expressed in its publications.

STREET PAVING CROWNS, WASHINGTON, D. C.

Discussion.*

By E. A. Steece, M. Am. Soc. C. E.

Mr. Steece.

E. A. Steece, M. Am. Soc. C. E. (by letter).—In the consideration of formulas for street paving crowns, as suggested by Mr. Powell, it is the writer's opinion that a complicated formula is of doubtful utility. The crown is made, primarily, for water parting, and, except for this, the advantage of the inclination formed at intersections, sharp curves, and possibly esthetic considerations, the roadway would have a plane surface transversely.

There have been, and, in all probability there will be, many suggestions for so-called rational formulas—differing somewhat from uniform ratios—for determining the crown. It appears that the first impulse of many highway engineers is to derive a formula to cover this vexed question; it also appears to be simple, at first glance, to write a good one, but the further it is developed the more unwieldy it becomes, until, finally, something almost as complicated as Kutter's hydraulic formula (if rational) is produced. Evidently, any formula which will not serve beyond the limits of common practice is not rational.

The formula attributed to Mr. Dare:

$$C = \frac{W\ (100 - 4\ P)}{6\,300 + 50\ P^2}$$

is a modification of that of the late A. Rosewater, M. Am. Soc. C. E.:

$$C = \frac{W\ (100 - 4\ P)}{5\,000}$$

*This discussion (of the paper by T. J. Powell, Assoc. M. Am. Soc. C. E., published in *Proceedings* for March, 1911, but not presented at any meeting) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

or *vice versa,* in which Mr. Steece.

C = the crown, in inches,
W = the width of the roadway, in inches, and
P = the percentage of grade.

Both these formulas produce lessening crowns on increasing grades.

The writer, in a limited practice, has observed that for impervious surfaces, such as asphalt, brick, etc., a crown of from 1 to 1.25% of the distance between curbs, for any grade, will come about as near to meeting the requirements of drainage and security to travel as any. For semi-pervious surfaces, which necessarily have an element of roughness, the crown should be made from 1.5 to 2.25% of the width of the roadway.

Such rules may be considered as too indefinite, but the engineer should exercise his judgment, as the conditions may warrant, for in this, as in every other branch of engineering, judgment is the prime necessity of good design, and cannot always be reduced to formula.

The writer is not inclined to view with much favor the plan of shifting the crown from the center of the roadway on streets having one curb lower than the other, as this not only extends the transverse slope on the lower side—augmenting an existing evil without accomplishing anything, except, in some instances, preventing the surface water from flowing across the roadway—but the unsymmetrical effect is unsightly.

The contention that the crown should be lessened on the steeper grades seems to be open to argument, also, notwithstanding the fact that eminent municipal engineers have decided that this should be done. To lessen the crown, in this instance, is to reduce the transverse slope to a minimum. Admitting that this feature is most jeopardizing to travel, it must be admitted, also, that it has its greatest effect where the speed of travel is greatest, which is most apt to be on the minimum grade. In cases where the crown is greatly reduced on steep grades, the gutters become too shallow to prevent storm-water from intersecting streets from flooding out to the crown and scattering débris over the roadway. While this is partly a storm-water and street-cleaning problem, these factors should receive their share of consideration.

If all elements are to be considered in determining street crowns, a degree of complication is produced which will hardly be susceptible of formulation. Withal, it is one of those subjects which are not small enough to leave nor large enough to bother with, but which recur just often enough to tantalize the engineering mind.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE GOING VALUE OF WATER-WORKS.

Discussion.*

By Messrs. Clinton S. Burns, Albert I. Frye, and Frederic P. Stearns.

Mr. Burns.

Clinton S. Burns, M. Am. Soc. C. E. (by letter).—The writer wishes to express his appreciation of the opportunity to read a paper so well prepared by men who have devoted much time and study to this comparatively new field of appraisement work—"Going Value."

It has been the writer's privilege to be associated with one of the authors in a number of appraisals of water-works properties, and he is fully in accord with the general views expressed in this paper, but in some particulars he must take issue, though it is with reluctance that he ventures to enter a discussion which may be somewhat at variance with the opinions of men who have so ably presented their subject. This discussion, however, is not to be construed as criticizing, or detracting from the merit of, the paper, but rather to add to its value, if possible, by pointing out some instances wherein its logic may be strengthened, or minor discrepancies may be eradicated.

The writer fully agrees with the authors in that going concern cost is as much a part of the reproduction cost of a public service corporation's property as is the reproduction cost of its physical plant. He is fully in accord with the statement that in reproducing a plant it is only proper and logical to consider what it would cost to rebuild it under present conditions; and that this applies to the business of the corporation as well as to its physical property. The writer likewise uses the method adopted by the authors, which they term the "Comparative Method."

A Problem in Futures.—It should be pointed out, however, that the

*This discussion (of the paper by Leonard Metcalf and John W. Alvord, Members, Am. Soc. C. E., published in *Proceedings* for February, 1911, and presented at the meeting of April 5th, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

determination of going value is a problem of the future, and therefore past or present income cannot be properly used as a basis of computation, except in so far as the past or present may be taken as a fair criterion of the future. The writer maintains, therefore, that the only basis on which going value may be computed, with absolute impartiality and fairness to both parties, is to determine first what is a fair rate in the aggregate, or a proper return on the fair value of the property, and then apply the comparative method to determine the going value. If the present income of a property is unreasonably low, due to rates which are insufficient, it is manifestly unfair to the company or corporation to compute the going value on that income; and if the present income is unreasonably high, it is equally unjust to the municipality to use that income as a basis of computation; furthermore, it is purely speculative to assume that the income will continue to be unreasonably high in the future, especially in view of the ever-increasing tendency toward the regulation of public service corporation rates. Therefore it is questionable whether the authors' definition of going value—"the cost of acquiring a given income"—is a logical one, unless it be first determined that the "given income" is a reasonable one, or a fair return on the fair value of the property. The fair value referred to must, of course, include the going value as well as the physical value. Mr. Burns.

Definition of Going Value.—In a discussion of the paper by H. E. Riggs, M. Am. Soc. C. E., entitled "Valuation of Public Service Property,"* the writer defined "Going Value" as:

"The present worth of the amount by which the anticipated profits of a going plant, operating at reasonable rates, exceed the present worth of the anticipated profits of a similar hypothetical starting plant [the comparative plant], operating at those same rates."

Further study seems to strengthen his faith in this definition as the logical conception of going value, as it seems especially well adapted to meet the requirement of fairness to both parties. If any public service corporation takes exception to it, it must thereby concede that its rates are unreasonably high. If the municipality takes exception, it thereby acknowledges that the rates are too low, or else confesses its desire to be unfair to the corporation.

At the Parting of the Ways.—The authors' illustration of the assumed capitalist, at the parting of the ways, is well presented, and, if their reasoning be pursued to its logical conclusion, there should be no confusion of ideas in the resulting "Going Value." They state, further, that this assumed capitalist, should he take the first path, and buy the existing plant,

"will evidently receive as return on his investment the net income indicated by its past history, modified by such circumstances as will

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

Mr. Burns. be likely to influence its net income during the period when a new comparative plant can reasonably hope to overtake its development."

This statement certainly must contemplate that there may be, and in fact are sure to be, some circumstances which will influence the net income of the future and may not be apparent from the past history of the property. Paramount among these circumstances is rate regulation, and, this being true, no prophecy of future net income is based on sound logic unless the possibility of rate regulation be considered. Consequently, the capitalist at the parting of the ways will certainly have cause to complain at his engineer's lack of judgment if this factor be overlooked in his computation of going value.

The Rate of Interest on Unemployed Capital.—The authors' statement, that "the larger the rate of interest allowed on the unemployed capital, the smaller the resulting going value," is apparently correct logic. However, the corollary of this statement would be as follows: If the unemployed capital is invested at a rate of interest equal to the net return from the existing water plant, then the going value reduces to zero. This phase of the problem merely emphasizes the correctness of the writer's contention that the income must be that due to reasonable rates. For it is manifest that, if the net income of the going plant is not greater than the return on unemployed capital, then the rates, in the aggregate, are too low, for no company should be required to perform a public service for less compensation than a proper return on capital invested in other pursuits involving the same degree of risk; and capital invested is entitled to larger returns than liquid capital.

It must be remembered that capital in liquid form produces no returns *per se* until invested in some productive enterprise, although the possessor of such capital may allow some bank or other investor to use it temporarily, keeping it in easily convertible invested form, and for this permission the bank or other investor pays to the possessor a portion of the profits accruing to the temporary investment, thus resulting in a low rate of interest.

Going Value Development Period.—The authors state that the past history of water-works, many of which were built during the Eighties, is not a fair criterion for the period required for the acquisition or development of business by water-works to-day. This appears to be in conflict with their prior statement on the same subject, but this latter statement is perhaps the result of more careful consideration and a deeper study of the subject.

The reason offered for this statement, that the public is now educated to higher standards of living, and to the use of water under pressure, which would result in much more rapid development of income by water-works now than formerly, is undoubtedly true; but this is by no means the principal reason. Of far greater influence is the fact that property is invested in house fixtures, plumbing, and in sewer connections.

Some experts maintain that were it not for the past existence of the water-works system, none of these vested property interests would be in existence, and that therefore these must be ignored in estimating the length of time required for the acquirement of the business. This, however, seems to be an assumption entirely contrary to pure logic, and not warranted by the facts. A corporation cannot take control of the vested property of an individual, nor claim any assets or benefits accruing therefrom. Hence, when considering the comparative plant, all property other than that belonging to the water company itself must be assumed to remain intact, and any computation of going value that fails to take these conditions into account is not in accord with correct logic. These plumbing fixtures, house connections, bath-rooms, sewer connections, lawns that demand water service, and all kindred metropolitan conditions that compel the uninterrupted continuance of the water service, are factors the existence of which cannot be denied. They are present actualities, and just as much to be considered as any local factor affecting the cost of reproduction of any of the physical property. Mr. Burns.

If, then, the appraiser is consistent in his logic, he must face these facts and give due consideration thereto in his estimate of going value, just as faithfully as he gives consideration to the local factors which affect the cost of duplication of any of the physical property. None of the theories advanced by the authors seems to cover the true situation, or to take into account all these factors so vital to a correct and strictly logical solution of the problem under discussion.

The Stage of Development.—Whether the plant finds itself in a stage of development termed "over-built," "normal," or "under-built," is recognized by the authors as a proper subject of inquiry in determining the going value. They state that the ability of the over-built plant to grow in income to its new capitalization cannot be recognized in computation of going value; but, in the case of the under-built plant, they state that the net income may be substantially reduced by the expenses incident to the additional capital required to bring the plant to a rational standard of water-works construction. This latter statement, when applied to their computation of going value, would, of course, substantially reduce it.

If it is true that the under-built plant should be brought to a rational standard, and that the computation to determine going value takes this fact into account by presupposing the normal condition to exist, then why not also treat the over-built plant similarly? Would not any other treatment be inconsistent with the logic? This again emphasizes the correctness of the writer's contention, that going value computation should be based on the theoretical net income which would accrue from reasonable rates, regardless of actual present income. Present income may be unreasonably high, or so low as to border on confiscation, and in neither of these situations would it be

Mr. Burns. fair or equitable to use actual income as the proper basis of computation. On the other hand, if a theoretically reasonable net income be assumed as a foundation for computation, the question of under-built or over-built condition is automatically adjusted in determining the proper net income to assume in the particular case under consideration.

Interest During Construction.—The authors have evidently fallen into error in their statement:

"Interest-during-construction payments are inevitable in all projects; * * * they constitute a forced expenditure, alike with the cost of the bricks and mortar, the pipes and the pumps, and cannot be recovered in any way except by addition to capital account or floating debt."

It is a little surprising that the authors should have become thus confused on the very point which they had just stated had been the source of some confusion in the minds of those who approach the subject for the first time. That they are in error is manifest by a moment's consideration of their original hypothesis of the assumed capitalist, at the parting of the ways, "having in hand the necessary capital." Now, a man who has the capital in hand, seeking an opportunity to convert it from liquid form to invested form, does not have any interest charges to pay during construction. He is constructing solely to provide a means whereby his idle capital may become useful and bring him returns. To make this point more evident, suppose that the assumed capitalist had in hand the necessary bricks, mortar, pipes, and pumps, and the necessary labor with which to construct the plant, instead of having in hand the necessary capital. By putting his idle labor at work and utilizing his otherwise useless materials, he may convert his property into a water-works plant which will bring him returns, but in so doing he does not incur any item of cost for lost interest during construction. Neither does the capitalist having in hand the necessary capital. Both are in exactly similar positions, seeking a means whereby interest may be earned on their idle property.

The lost interest during construction, therefore, must of necessity appear as an element of the going value, by the comparative method, and to include it also in the capital account charged against construction of the physical plant is a duplication of accounts. If their original hypothesis had assumed a penniless promoter, at the parting of the ways, with ability to borrow the necessary capital, lost interest during construction would then be a proper item to charge in the capital account of the comparative plant, but in this event the going plant would also have interest chargeable against it. The error seems to be that they have treated the comparative plant as having to be constructed by the penniless promoter, then this promoter is miraculously converted into a capitalist, without having bridged the

chasm between promoter and capitalist—popularly supposed to require a bridge of no mean proportions. Mr. Burns

Application of Principles.—This duplication of accounts is clearly shown in Table 3, wherein $482 000 is stated as lost interest during construction, while Table 4 shows the total net profits of the existing plant. The capitalist at the parting of the ways, with capital in hand, had he taken the first path and purchased the existing plant, would have come into the immediate enjoyment of the net returns set forth in Table 4; but, had he taken the other path and elected to construct the "comparative plant" he would not have lost this $482 000, designated as interest lost during construction, but instead thereof he would have lost the opportunity to realize the greater net return shown in Table 4. He could not include any of this $482 000 as an item of expense in the capital account of the comparative plant, because, by the fundamental hypothesis of the comparative method, his capital is not earning, but is seeking the opportunity to earn. He could not lose interest that he did not possess.

Conclusion.—Going value is a subject which has but recently received scientific study and careful consideration from engineers and appraisers. Any method advocated for its determination, therefore, must of necessity pass through the successive stages of development so essential to the perfection of any new subject, and it is only by earnest study, honest effort, and fair-minded discussion of such papers as this, that true progress can be attained. It is largely an exercise in pure logic, and, therefore, it is not surprising that some erroneous conclusions should have been developed, based perhaps on theories springing from illogical assumptions. It is with a keen appreciation of these conditions that the writer offers this discussion on a subject second to none in its demands for clear thought, deep logic, and correct conclusions.

Any theories advanced by the writer are offered to invite discussion, in the hope that this much complicated subject may thereby be freed from some of the confusion with which it has become enveloped.

ALBERT I. FRYE, M. AM. SOC. C. E. (by letter).—This able paper brings out a new point to be considered in connection with the valuation of water-works, namely: Shall the going value be considered as the actual going cost, economically administered, in building up the business of the existing plant? or shall it be considered as the estimated going cost necessary in developing the business of the prospective duplicate plant? Mr. Frye.

The writer believes that before a fair valuation of the works can be made, the appraisers should gather and formulate the following data:

Aa. The actual cost of the existing physical plant;
Ab. The actual going cost in connection with *Aa;*

Mr. Frye.

Ba. The estimated cost of the duplicate physical plant;
Bb. The estimated going cost in connection with *Ba;*
K. A curve or table showing the annual profits, in percentages, on the total investments.

Under normal conditions of good management, the above going costs may be considered as going values; otherwise they may be reduced to going values by possible elimination of excessive or unnecessary items.

The basic cost of the project would then be considered to be either, $C = Aa + Ab$; or, $C_1 = Ba + Bb$; or, some compromise between the two; and the appraised value of the project should be based on all three of the items: $V = Aa + Ab$; and, $V_1 = Ba + Bb$; and, $V_2 =$ the average of the last few years' profits capitalized at the usual industrial rate of interest, say 6%, more or less.

These should be weighed and balanced very carefully, and it is of course understood that interest is to be included in each of the above cases.

The contention of the authors that the estimated going cost, *Bb,* rather than the actual going cost, *Ab,* should be associated with the estimated cost, *Ba,* in arriving at a fair appraisal value, is, in the writer's opinion, a tenable one.

Mr. Stearns.

FREDERIC P. STEARNS, PAST-PRESIDENT, AM. SOC. C. E. (by letter).—At the present time, the growing prominence of matters relating to the valuation and rate regulation of public service properties warrants a full discussion by engineers of all questions relating thereto, and the authors of this paper have presented a valuable contribution on the subject.

It may be well to state in the beginning that the writer has been connected with the valuation of water-works property for the purposes of sale and rate fixing, and although the principles involved in such valuations are in the main the same as in those of other public service properties, they necessarily differ in detail.

The term "going value," or "going concern value," is an unfortunate one, because it may be given different definitions. The authors have defined it "as the value of a created income, or—from the reproduction point of view—the cost of acquiring a given income." They also speak of it as "the investment necessary to put the plant into successful operation and to create the revenues that justify its construction," and, after this statement, say: "This portion of the cost of a project has been termed by the Courts, the 'going value,' or the 'going concern value.'"

In many cases, engineers and the Courts have referred to the "going concern value" in such a way as to show clearly that they do not have in mind the definitions above given; for instance, Justice Moody, of the Supreme Court of the United States, in the Knoxville case, referring to an allowance for the "going concern," says:

"The latter sum we understand to be an expression of the added value of the plant as a whole over the sum of the values of its component parts, which is attached to it because it is in active and successful operation and earning a return." Mr. Stearns.

The Judge of a United States Circuit Court in California, in rendering an important decision, less than three years ago, said:

"The value of the franchise and going business depends upon their earning power."

He also said:

"A plant with an established business, with customers who have connected their houses with the Company's distributing pipes is more valuable than it would be without such connections and without such customers."

When there is competition, there may be a "good-will" value connected with a going concern which would not exist if the concern were not a going one, and this part of the value of the going concern would be based on the earnings and the degree of success which has been attained; but when the public service corporation has a practical monopoly and the commodity sold is an absolute necessity—as, for instance, water to the inhabitants of a large city—it seems clear that, under such circumstances, there can be practically no "good-will" value to be included in the "going value."

This is the view expressed by the United States Supreme Court in the Consolidated Gas Company case, where it stated:

"We are also of opinion that it is not a case for a valuation of 'good will.' * * * The complainant has a monopoly, in fact, and a consumer must take gas from it or go without. He will resort to the 'old stand,' because he cannot get gas anywhere else. The court below excluded that item, and we concur in that action."

The writer believes it would aid the Courts and others if a new term were used for that portion of the going value which represents the special expenses and risks connected with the construction of the works, and the expense of developing the business of the original works and of additions thereto. The propriety of including in this portion of the going value other sums than those relating to the development of the business will be discussed later.

Halbert P. Gillette, M. Am. Soc. C. E., in his recent discussion of Mr. Riggs' paper, "The Valuation of Public Service Corporation Property,"* has used the term, "development expense," in the same sense as the authors have used the term, "going value," and this or some similar term seems less likely to cause confusion than the term, "going value," which, under some circumstances, has a more comprehensive meaning.

It seems to be generally admitted that the Courts favor the repro-

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 1.

Mr. Stearns.

duction method of ascertaining the physical value of property, and that they do this, especially in the case of old property, first, because there may be marked changes in the value of real estate and the cost of works from the time they were built to the time when they are valued, and it is the present and not the past value that is desired; second, because of the difficulty of proving the legitimate cost of works constructed many years before, on account of defective bookkeeping and illegitimate and extravagant methods of construction; and third, on account of the abandonment of parts of the work which were included in the original cost.

There is no reason to believe that, with normal works constructed in a legitimate manner, with efficient bookkeeping, and without change in the prices of real estate, labor, and materials, the Courts would prefer the estimated to the actual cost of the works, and it also seems reasonable to believe that they would not hesitate to include the total outgo for the construction of the works and the development of the business, even though there were included in the accounts the cost of a reasonable proportion of temporary and unsuccessful works which had been abandoned and therefore would not appear in the schedule used in determining the cost of reproduction.

The writer, therefore, believes that, in determining the reproduction value of the tangible and intangible elements connected with the development of normal works and of the business of such works, the engineer should endeavor to obtain results agreeing as closely as possible with the probable outlay if the works in question had been built and the business developed when the value of real estate, the prices for labor and materials, and other circumstances, were the same as at the time of valuation.

Assuming the above view to be correct, does the authors' method of ascertaining the "going concern value," defined as the cost of developing the business, represent the probable outlay for such development under similar circumstances as to prices and otherwise?

Applying the rule to water-works, it is the writer's judgment, that, in the case of the works first introduced for the general supply to a city or town, it does apply, and gives a development expense which would be in fair accord with the actual outlay for developing the business.

He is as firmly of the opinion that, in the case of the water supply of a large city which long ago outgrew the capacity of its original works and is supplied mainly by works which have been added from time to time, the method used by the authors will give a result wholly out of accord with the actual outlay for developing the business of such works, even after making due allowance for difference in prices and in other conditions at the time when the outlay was actually made and at the time of valuation.

Mr. Stearns.

It is clear that when works are first introduced into a city or town where the water supply is obtained from wells by inhabitants unaccustomed to a public water supply, and where for several years the demand for water is likely to represent but a small part of the capacity of the original works, the cost of developing the business is likely to be a considerable percentage of the cost of the works; whereas the cost of further developing the business to the extent required to provide a return on expenditures for additions to the works will generally be a much smaller percentage of the cost of such additions. The latter statement will be especially true in cases where the return which the public service corporation is permitted to collect is based on the physical valuation of the property and is determined at frequent intervals.

For instance, the provisions of the Constitution of the State of California require that the rates or compensation to be collected for water supplied to any city or town shall be fixed annually by the governing body of such city or town. In fixing such compensation, the physical valuation of the property is generally a factor, and, in accordance with certain legal decisions in that State, it is usual to exclude items of property under construction up to the time when they are put in use, and then to include them as a portion of the operating plant. Additions to a normal plant, therefore, become productive property on which the same rate of return is allowed as on the other property at the time of the next rate fixing. That is to say, the cost of developing the business for the additions to the works is substantially the loss of interest on their cost for a period, averaging less than a year, extending from the time when they are put in use to the time when they are included in the physical valuation.

Nothing in the authors' description of their method of determining the "going value" or "development expense," or in the example illustrating their method, indicates that the results obtained in the case of a large and old plant will be any smaller percentage of the value of the property than in the case of a small plant, although, in accordance with the views set forth by the writer, the cost of developing the business for the additions which make up the greater part of a large plant is a much smaller percentage than for the original plant.

Mr. Alvord has given a table* containing certain data of the valuations of water-works properties, including the going value attributed to such properties. If one takes the data from this table, omitting the places having less than 10 000 inhabitants and the one place having a population of more than 100 000—the former being omitted because of their small size and the latter because it is hardly possible that it could be supplied from its original works—and also omits one

* *Proceedings*, American Water-Works Association, 1909, p. 206.

Mr. Stearns. case where there was competition between two rival plants, the following minimum, average, and maximum results are obtained:

Average population		34 555
Average net value of physical property		$774 567
Average going value		$133 269
Going value per capita:	Minimum $1.67	
	Average 3.86	
	Maximum 6.90	
Percentage of value of net physical property represented by the going value:	Minimum 10.5	
	Average 17.2	
	Maximum 34.6	

Mr. Alvord calls attention to the lack of relation between the going value and the value of the physical property, and yet an average of the results attained in a number of cases where rational methods were used will give approximately this relation in the case of a normal plant.

In the typical illustration used by Messrs. Metcalf and Alvord, the statistics are as follows:

Population	300 000
Net value of physical property, including interest during construction	$9 082 000
Going value	$1 897 000
Going value per capita	$6.32
Percentage of value of net physical property represented by the going value	20.9

The writer can easily conceive that there may be an actual deficiency in the returns during the earlier years of the operation of a normal water-works plant which will equal $133 000 for a plant valued at $774 000 and supplying 34 000 people, but he cannot conceive that during the growth of such a plant to a net value of $9 082-000, with ability to supply 300 000 people, that there can be an added deficit amounting to $1 764 000, or any sum approximating this amount, due to the development of the business incidental to the additions made from time to time.

As already stated, he believes that no method will be satisfactory that does not produce results which are in fair agreement with what actually happens in the operation and construction of works. The assumption that a large city, say one having a population of from 100 000 to 1 000 000, is without a water supply and that a new supply is to be introduced, is one which has no parallel in actual practice, and as the results based on this assumption differ greatly from those of actual experience on works which have been developed to supply these large cities, the results for such populations are unsatisfactory.

The writer has already stated that, in addition to the expense of developing the business, there are other expenses and risks in actual practice for which the owner of public service property should be

compensated, and which do not appear in the ordinary reproduction value. This view may best be explained by giving some instances. Mr. Stearns.

If one were to value the Quebec Bridge after it has been completed and successfully tested, he would not think it proper to ascertain the cost of the bridge which failed and add it to that of the successful bridge, but, in the writer's opinion, it would be proper to add to the cost of the successful bridge a sum representing the value of the risk incurred in building a work of such unusual magnitude. In other words, he believes that if the company which is now building this bridge could know in advance exactly what it would cost for construction, engineering, interest during construction, and all other items of expense, it would willingly pay a considerably larger sum for the bridge constructed and successfully tested and of assured stability.

Take another instance, such as the building of a large earth dam having a height of 100 ft. or more. Such dams are frequently built by first constructing a tunnel through an adjoining hill, or by laying a pipe or conduit through the dam, to take care of the flow of the stream during construction. The capacity of such conduits is generally too limited to carry the discharge of large floods. Dependence, therefore, is placed on building the base of the dam during the dry season and on utilizing the storage to assist the conduit in taking care of the floods. In such construction, however, there is a risk that an exceptional flood may come, after the work is well advanced, and destroy it. On the other hand, all may go well until the completion of the work, in which case the dam, in the writer's opinion, is worth more than the cost of construction, by the value of the risk incurred in connection with its construction.

In one case where the writer was engaged with others in the valuation of a masonry dam 222 ft. high, it was originally planned to build a rock-fill dam, and its construction proceeded until it had been built up 54 ft. above the level of the stream, when an exceptional flood removed the whole of the rock-fill in a single night.

The rock-fill dam necessarily did not appear in the schedule of property valued, but, had it been completed and proved successful, it would have been worth more than its cost on account of the risk incurred in its construction.

Accounts are frequently seen in engineering papers, or are the results of personal observation, where dams and other structures have failed to stand the test of use, or where incidents connected with the construction of works have caused an expense far beyond what is provided for under the head of contingencies.

As a few instances, the writer will cite the slip at the Necaxa Dam, the encountering of bad ground in the Loetschberg Tunnel, in Switzerland, which required the abandoning of about a mile of the

Mr. Stearns. tunnel and the diversion of the line through another portion of the mountain, and the change of plan of the new Croton Dam of the City of New York, which involved an expense of more than $1 000 000 for construction and interest, which would not appear in the reproduction value of that property.

These examples might be extended in many directions, to show that public service corporations and cities, without negligence on their part, may expend large sums for works which fail or otherwise are unsuccessful, and for which they will not receive credit in the reproduction value determined in the usual way.

It may be questioned by some whether failures and unsuccessful works are not covered by the allowance for contingencies, which is generally included in the reproduction value of the physical property and must be included in any preliminary estimate of the cost of work to be done, if the engineer intends to have his estimate agree with the probable cost of the completed work. There is no doubt that a sufficient allowance for contingencies would cover also the risk of failures and non-success, but the writer does not believe that it is the custom of engineers to make sufficient allowance for this purpose in works of a hazardous nature.

It would not be fair to the public to charge to the value of the works the amount of money lost by some great disaster, but on the other hand, it is not fair to the public service corporation constructing hazardous works to charge only the cost of reproducing such works, without adding to such cost the value of the risk which they have incurred. This value the writer would include as a "development expense" and, consequently, as a part of the "going concern value."

In the early years of any large plant, defects in design or apparatus are frequently discovered, and are made good at a considerable expense. Temporary works are frequently installed, and are given up as soon as the permanent works are completed. All such items of expense do not appear in the reproduction value of the physical property, and, unless otherwise provided for, should be included as a part of the "development expense."

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PAPERS AND DISCUSSIONS

DAMS ON SAND FOUNDATIONS: SOME PRINCIPLES INVOLVED IN THEIR DESIGN, AND THE LAW GOVERNING THE DEPTH OF PENETRATION REQUIRED FOR SHEET-PILING.

Discussion.*

By Messrs. W. G. Price, Alexander Potter, T. Kennard Thomson, G. E. P. Smith, Allen Hazen, and R. C. Beardsley.

W. G. Price, M. Am. Soc. C. E. (by letter).—The writer has learned by experience that it is possible to construct on a sand foundation a dam which cannot be carried away by the force of flood-waters, although the cost will usually be greater than when there is a rock foundation. Mr. Price.

The design must be such that parts of the dam cannot be separated from the main part by any force of water which will come against it. The whole structure must be heavy, and so broad that it cannot be moved bodily down stream, or be overturned. It must be very flexible throughout, so that when the water cuts the sand or gravel out from under it, the dam will bend and stretch, and sink into the cavity without breaking, and the material which goes into the cavity must then be replaced with the same kind of material on top of the dam; and this must be repeated until the force of the water can dig no deeper.

If the banks of the stream are also composed of material which can be eroded, the structure must be carried far enough back and over the bank, and made of sufficient vertical thickness to enable it to drop flexibly and fill any cavity which may be cut out under it.

* This discussion (of the paper by Arnold C. Koenig, Assoc. M. Am. Soc. C. E., published in *Proceedings* for January, 1911, and presented at the meeting of March 15th, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Price. There is a limit to the depth to which the force of the water can undermine and sink a structure of this kind, and, when that limit has been reached, the top part of the dam, or that part which is exposed to the air, can be finished in concrete.

Sheet-piling may be necessary; but there must be no piling under or attached to the dam, which will prevent it from sinking.

The writer has built a number of dams on this plan, where the sand was too deep for a rock or other secure foundation to be reached, and they have not been carried away. He constructed the two dams in the Atchafalaya River at Simmesport, La., the type being conceived and designed by him, as above described. They were completed in 1890, and are still in good condition.

Under these dams there was a stratum of fine sand 80 ft. thick, and it extended for miles on each side of the river. The depth of the river at low water was about 50 ft., and the dams in the center were depressed to 5 ft. below low water. During floods the river rises 50 ft. above the dams, and the velocity of the water directly over them is then more than 20 ft. per sec., and the discharge nearly 500 000 cu. ft. per sec.

It is the writer's opinion that permanent dams on this plan can be constructed across any alluvial stream, such as the Missouri, Mississippi, or the Colorado, with its bed of very fine silt. The water may dig 50 ft. or more in depth before it gives up the fight to destroy the dam.

Mr. Potter. ALEXANDER POTTER, ASSOC. M. AM. SOC. C. E. (by letter).—By the census recently published, many cities in the Western States show increases of from 300 to 500% in the last decade, and in many cases there is nothing to prevent their continued growth. This condition of abnormal growth also exists in the Canadian Northwest. To meet the increasing municipal and commercial needs of these cities, it is becoming necessary to harness the rivers and streams, but on many of them it is impossible, and on others impracticable, for economic reasons, to found the necessary structures on rock. This paper, therefore, is timely, and it is to be hoped that those who have had experience in founding large structures of this type on what are considered to be unstable foundations, will relate the results of their experience.

On many of these Western rivers it becomes necessary to construct dams which do not extend across the full width of the permanent valley. The alluvial deposits often form a natural dam for the greater part of the total width of the original valley, and the rate of percolation through this natural dam becomes an important feature in the economy of the design. In such cases the determination of the rate of percolation, therefore, is also important.

It will be conceded that the main problem is to prevent even the

slightest movement in the material on which the dam is founded, and it is pleasing to note that the author's experience with quicksand as a foundation warrants his conclusion that it has lost the terrors that it ordinarily arouses. Mr. Potter.

The writer was recently forced to face the problem of designing a dam under just such conditions as are referred to by the author, and he was impressed by the lack of precedent and information concerning such an important branch of engineering. Therefore, he voices the hope of the author that this paper will bring out a full and free discussion from those who have had experience in the construction of dams which have not been founded on solid material.

For the depth of penetration of sheet-piling the author suggests a general rule which presupposes the ability to procure a continuous line of water-tight sheet-piling to the depth he mentions. For high dams, wooden sheet-piling is out of the question, because of the inability to drive it when the dam is founded on material requiring a great depth of penetration. Even with dams of less height, the difficulty of securing a continuous line of tight sheet-piling is great, and the cases on record where a continuous and tight sheet-piling has been driven to a depth of 30 ft. or more, are few.

The use of steel sheet-piling for a cut-off wall of a reservoir may be open to criticism because of the possibility and probability of abrasion of the protective coating on the steel in the process of driving. The extent of the deterioration of the steel when used for such construction is of great importance, and the writer has found it difficult to secure any reliable evidence as to the lasting qualities of steel when used under somewhat similar conditions. Contributions from engineers having experience with steel sheet-piling, under conditions which could be taken as precedent for a steel cut-off wall, should form a very valuable feature of the discussion of this paper.

The writer's attention has recently been called to a process of constructing a continuous concrete cut-off wall of any depth by first driving square timbers to which are fastened steel channels. The timbers between the channels are removed one at a time, the channels being left in place for forms, and the space between them filled with concrete. When the concrete has set, the steel channels are removed. The writer trusts that some one familiar with this method will describe it.

The author's remarks on the percolation of water through sand and their application to show one of the principal causes of failure of sheet-piling dams are very interesting. Attention might be called to another cause of failure of such structures not founded on rock, and one which has not received the study it merits. In a structure of this type of any considerable magnitude, it is impossible to prevent unequal settlement. Such settlement is due principally to the non-homogeneity of the compressible foundation; sometimes, in a

Mr. Potter. poorly designed structure, it is due to the excessive foundation pressures developed at various points. Provision should be made for unequal settlement in such a structure, as it is often sufficient to rupture it completely at one or more points. These settlement cracks, often of considerable magnitude, are very serious when they develop in the lower part of the dam. A small stream of water begins to percolate through the crack, and its tendency is to undermine the foundation gradually by removing the fine material. This undermining and settlement once begun, is progressive, and the ultimate failure of the structure is only a question of time.

Settlement joints—or, as they are sometimes erroneously called, "expansion joints"—should extend entirely through the structure, and permit of a liberal movement in any direction at the joint. There appears to be no reason why these joints cannot be made permanently water-tight with either lead or copper plates. The distance between such joints is at present an open question, but it should not exceed 200 ft.

Unequal settlement and percolation under a dam can be prevented to a very great extent by increasing the width of the base, within reasonable limits of economy. The width of the base adds to the stability far more than any other single factor.

Attention is also called to the fact that the author has not brought out with sufficient force the necessity of relieving, by a system of drains, the hydrostatic pressure constantly developing under the floor of the dam. The necessity of relieving this pressure as much as possible is as great, for the comparatively narrow, solid, gravity section, as for the hollow, reinforced concrete dam, if not greater.

Mr. Thomson. T. Kennard Thomson, M. Am. Soc. C. E.—If a rock foundation cannot be obtained, either because there is no rock or on account of inability to raise funds for a permanent dam, a dam on sand foundations may be quite justified, provided it is not made too high. There is no doubt that sheet-piling would greatly retard the flow of water under a dam, but it would be very dangerous to assume that the results of this retarding could be determined definitely by calculation, for, as the author very properly says, the failures are more often "on account of carelessness in driving and faulty alignment than from insufficient depth of penetration." This carelessness or unintentionally poor work, however, is always likely to occur. Cases are well known where piles, driven with considerable care, have afterward been found to have been telescoped or badly bushed, without having gone down to the proper depth or as deep as supposed. Many have seen wooden sheet-piling, driven with the utmost care, with tongued and grooved or similar joints, which appeared to be in perfect alignment on the surface, and yet the piles have been forced apart below ground. In the Brooklyn Navy Yard some steel sheet-piling had been, apparently, per-

fectly driven, the tops showing in true alignment, but the bottoms were twisted, bent, and coiled in all imaginable shapes; and yet the piling consisted of 8- and 10-in. channels separated by 6-in. timbers and firmly bolted together. It would be difficult to get a stronger sheet-piling. Mr. Thomson.

For these reasons, the speaker would not like to rely on driving sheet-piling to such a depth as 50 ft., as suggested by the author; for, if the conditions were such that one could be sure of driving it to such depths, the ground would be so soft as to be unsafe. As it is impossible to ascertain the condition of driven piling, one must take a good deal for granted. The speaker does not, by any means, wish to infer that he would not recommend sheet-piling to retard the flow; he simply wishes to utter a warning against placing too much reliance on such protection.

The sections, shown by the author—Figs. 1, 2, 3, and 4—for concrete dams on a sand or earth foundation, would be sure to develop bad cracks. In fact, the speaker does not consider such masonry dams at all appropriate for a sand or loam foundation, as the foundation is more or less yielding, while the structure above is naturally unyielding, until collapse. It would seem that an earth dam with a core-wall would be much more appropriate. The sheeting could be driven under or near the core-wall.

G. E. P. Smith, Assoc. M. Am. Soc. C. E. (by letter).—Dams on sand foundations are now quite generally thought to be as necessary engineering projects as those on rock bottoms. The examples of such structures now in successful use show wide variation and also great ingenuity in their design. There have been failures of such dams, usually due to unprecedented floods for which the dams were not designed, or to the lack of engineering direction in their design or in their construction, or to the slow deterioration of some portion of the structures without attention to their up-keep. Any of these causes may be equally disastrous to dams on bed-rock. Mr. Smith.

As a preliminary to the discussion of dams on sand beds, the laws of flow through saturated pervious deposits should be understood. The following remarks have reference to those cases in which it is impracticable to shut off the underflow of water by a curtain-wall extending to an impervious stratum.

The resistance to seepage flow is that due to water friction, both internal and surface, and depends therefore on the character of the medium and, to a lesser extent, on the viscosity of the water. Capillary attraction does not exist as a force in saturated deposits. It has been shown by investigators that the velocity of seepage flow is a function of the porosity of the material, of the "effective size" of grains, of the hydraulic gradient, and of the temperature, each in an important degree.

Mr. Smith. The "effective size" of a sand was defined by Hazen as that size, than which 90% by weight of the material is coarser and 10% is finer, a definition based on experiments with Massachusetts deposits. The resulting formula coefficients, however, are not of wide application, and have been shown to be much in error when applied to river sands in the Rocky Mountains. Grading sands with King's aspirator is a more rational method, but in this case, also, the coefficient in the flow formula and the graduation table are dependent on the sand used in the rating. No satisfactory basis has yet been found for classifying sands and gravels. The effects of the sharpness or roundness of grain, of the various crystalline constituents (for example, mica), and especially of the peculiar stratification of sand *in situ* due to its sorting by running water, seem quite incapable of mathematical expression. Therefore the formulas for seepage flow are useful for deducing comparative results in a region, rather than for obtaining absolute values. A recent article in the technical press, describing the construction of weirs on sand foundations in India, proposes empirical formulas for general use in designing such structures. It is possible that the formulas should be much modified for use in America, or rather, in each section of America.

It is to be noted that the velocity of true seepage flow varies approximately as the first power of the pressure head, or of the gradient, and hence the ordinary formula for free flow,

$$v = \sqrt{2\,g\,h},$$

is inapplicable to this discussion. With very coarse sorted materials, the flow is not exactly seepage, but is partly free flow.

The flow through river sands *in situ* is of greater interest than that of flow through screened media. Much experimental knowledge of the former has been obtained by Professor C. S. Slichter, who devised an electrolyte method of measuring the rate of movement of ground-water, and has carried on investigations in many localities, from Long Island to California. His results are available in the Water Supply Papers of the U. S. Geological Survey. The average velocity of underflow in the Arkansas River sand was found by Slichter to be 7.4 ft. per 24 hours, while the maximum value obtained was 22.9 ft. The latter value is equivalent to a velocity of about $\frac{1}{8}$ in. per sec. with a hydraulic gradient of 1:10. The writer found maximum velocities of underflow of more than 400 ft. per day on an Arizona stream with a gradient of 20 ft. per mile. Before designing a dam or weir on a sand foundation, where the perviousness of the sand is unknown, this factor should be determined by direct measurements.

The effects of seepage flow beneath a structure are of two kinds: first, the grains of the material may be moved, particularly at the toe of the apron, where a point of maximum seepage velocity exists;

second, the distribution and amount of upward pressure on the structure is modified by the seepage, and may be the primary cause of failure, especially if there is a wide apron. The first effect depends on the velocity of the seepage and the size of the grains of sand, and may be made inactive by lengthening the path of the overflow. The danger is reduced by placing a rip-rap of boulders at the toe, the smallest ones at the bottom, the largest ones on top. The second effect is dependent on the design of the foundation and base of the structure. Mr. Smith.

In his treatment of sheet-piling, the author has endeavored to establish a relationship between the depth of penetration and the head of water behind the dam. It would seem, however, that the piling should

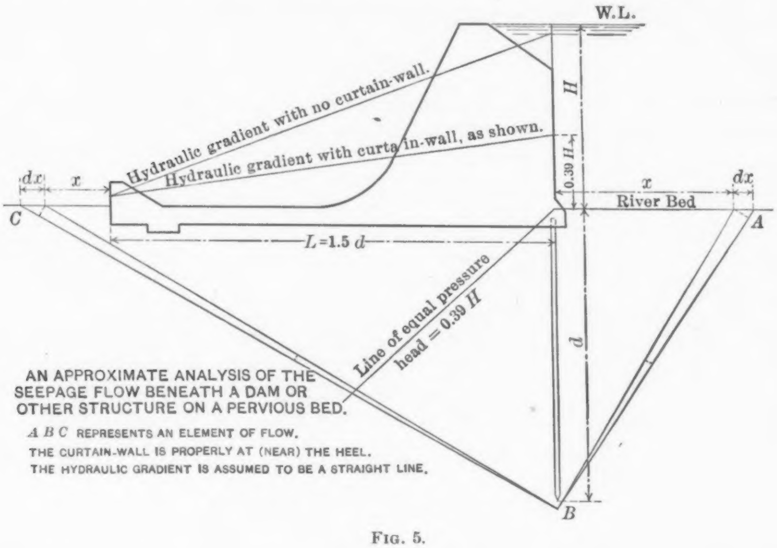

Fig. 5.

not be considered as an independent feature of the design. The same result can be attained by a long base with even no piling at all, or by a short base with piling, and the choice of the final design is largely an economic one. In the Laguna Dam very short piling was used, and its effect in "discouraging" the underflow of water is not great. In that case rock filling was obtained cheaply, and the wide base was adopted.

To illustrate the effect of a curtain-wall on the stability of a structure, and to show the general dependence of the problem on the law of seepage flow, the following approximate analysis is made. In Fig. 5 the ratios of base, head, and depth of penetration are retained as in Fig. 2, but the figure is simplified for the purpose of analysis.

Mr. Smith.

Let H = maximum head of water between heel and toe of dam, usually occurring at low water;

d (= — $1.6H$) = depth of penetration;

L (= $1.5d$) = length of base;

h' = head lost on the right of the curtain-wall;

h'' = " " " " left " " " "

C = a constant, depending on the character of the sand;

Q = quantity of seepage flow per unit of time.

Let ABC be a differential element of flow. The assumptions as to the length of the element and its triangular shape are approximate, and introduce errors, which, however, tend to balance each other. The sand is assumed to be uniform in character.

The formula for seepage flow may be written thus:

$$Q = C\frac{H\,S}{L},$$

in which S is the cross-sectional area. In the differential triangle, on the up-stream side, the average cross-section is

$$S = \frac{1}{2}\frac{d}{\sqrt{d^2 + x^2}}\,dx.$$

Then

$$dQ = \frac{Ch'\,d}{2}\frac{dx}{d^2 + x^2};$$

$$Q = \left[\frac{Ch'}{2}\tan.^{-1}\frac{x}{d}\right]_o^\infty = \frac{1}{4}\pi\,Ch';$$

$$h' = 1.27\,\frac{Q}{C}.$$

Similarly, on the down-stream side

$$S = \frac{3}{4}\frac{d}{\sqrt{x^2 + 3dx + 3.25d^2}}\,dx.$$

Whence

$$h'' = 2.3\,\frac{Q}{C}.$$

Therefore, about one-third of the head, H, is lost, or neutralized, on the up-stream side of the piling. The upward pressure at the heel of the dam is not measured by the pressure at the point of the piling. There is a further loss of head in the flow upward, and evidently it may be taken at two-fifths of h'', so that the upward pressure at the heel is equal to three-fifths of h'', and hence equals $0.39H$. This heel pressure is shown in Fig. 5 as the ordinate used to determine the hydraulic gradient. The upward pressure decreases from the heel to a small value at the toe. The gradient is subject to a correction dependent on the depth of the base below the river bed.

As stated above, this demonstration is illustrative merely, but it exhibits the principle that the problem involves all the lines of flow. To consider only that line of flow which follows along the piling and the base of the dam is erroneous. Mr. Smith.

Without the curtain-wall, the hydraulic gradient would be the higher line shown in Fig. 5, allowance being made for losses at entrance and exit. Therefore the effect of the curtain-wall, for the given ratio of base to depth of penetration, is to reduce the upward pressure on the structure by about one-half. The velocity of flow will also be diminished about one-half by the curtain-wall, inasmuch as the minimum length of the flow line is increased from 1.5*d* to 2.8*d*.

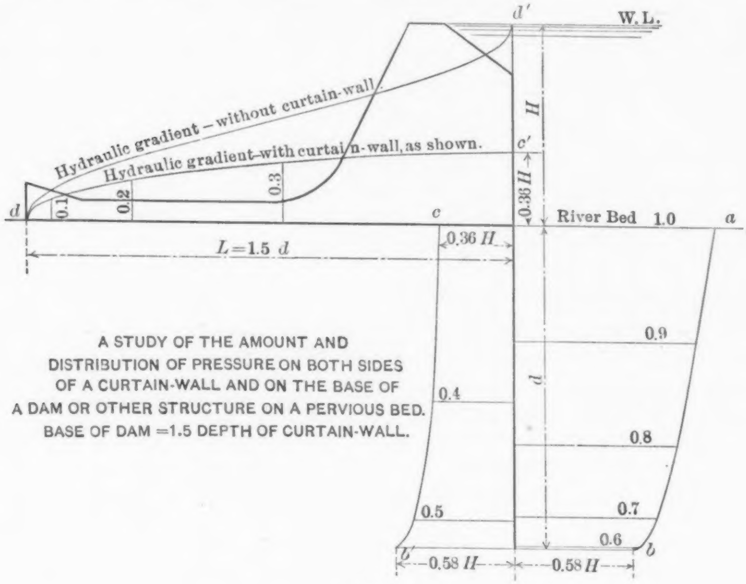

FIG. 6.

The effect of another line of sheet-piling placed near the toe of the structure would be to reduce the velocity of the underflow, but to increase the upward pressure. Sheet-piling is most effective when concentrated at or near the heel, and piling which is water-tight should not be used under the toe or beneath the apron. Referring to the detached line of piling and concrete curb, as proposed by the author and shown in Figs. 1 and 2, would not the effect be to increase the upward seepage flow between the curb and the toe of the dam, and thus be an element of weakness rather than safety? In some of the rivers of the West—the Gila, for example—detached piling of that character is apt to disappear during high floods.

Mr. Smith. At the writer's suggestion, Professor H. C. Wolff, of the mathematics department, University of Wisconsin, has undertaken the rigorous solution of the problem of underflow beneath a dam. It involves the use of a pressure function which is determined to satisfy Laplace's equation, with the imposed boundary conditions. The case is entirely analogous to problems of the flow of heat.

The results of Professor Wolff's analysis, applied to the structure discussed above, are shown in Fig. 6. The flow boundary is down the face of the piling, up the back, and along the base of the dam. The head is considered to be divided into ten equal parts, and the losses

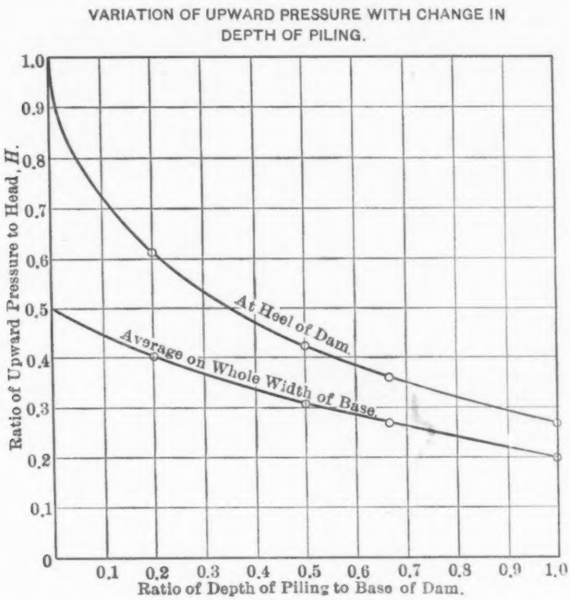

FIG. 7.

of the successive parts are indicated by the ten divisions along the boundary. The hydraulic, or pressure, gradient is then plotted in the lines, *ab, b′c,* and *c′d.* If H is assumed to be 10 ft., then each ordinate is shorter than the previous ordinate by 1 ft. The upward pressure on the dam varies from $0.36H$ at the heel to no pressure directly at the toe; but, at a short distance back from the toe, the pressure on the apron is considerable. The average upward pressure on the base is $0.27H$. Relatively high velocities of seepage flow occur at the point of the piling and at the toe of the dam, as indicated by the slope of the hydraulic gradient.

The line, *d'd*, represents the hydraulic gradient when there is no curtain-wall beneath the dam. The line is plotted from thirty-five computed points. It is a sine, or ogee, curve, and the maximum velocities occur at the heel and at the toe. The average upward pressure is $0.5H$. At one-tenth the base length from the toe the pressure is $0.21H$. Mr. Smith.

By applying the same method of analysis to a series of cases, in which the depth of penetration is taken at various percentages of the base, the curves of Fig. 7 are obtained. The curves show how the average upward pressure and the upward pressure at the heel of the dam decrease as the depth of the piling increases. It is apparent that the benefit derived from the piling decreases with the depth—an example of the principle of diminishing returns.

It appears, also, that the uplift on the apron is much greater than it is ordinarily assumed. Aprons which are very thin in section are common. Much thought must be given to the design of the apron and of the overfall face.

The entire problem of dams and weirs on sandy river beds is one of unusual difficulty, and is complicated by uncertain factors such as the effects of impact and cross-currents, and the extremely variable velocity of seepage in different sands. Doubtless our knowledge concerning such structures will be greatly increased in the future, as their number increases.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—An investigation of the stability of dams built on sands and gravels is a most commendable undertaking, and is likely to lead to success in many cases. In reaching success, an adequate knowledge of the laws of the flow of water through sands and gravels is of fundamental importance, and the writer regrets to see such calculations discarded by the author. Mr. Hazen.

The results credited to the Massachusetts State Board of Health were the work of the writer, about twenty years ago, at the Lawrence Experiment Station.*

These results have been quoted many times, but in doing this their form has been sometimes changed, new assumptions have been introduced, and limitations originally made have been omitted, so that, in using them at the present time, the only safe way is to refer to the original publication.

There is no reason why flows through gravels should follow the formula given for sand, as the author seems to suppose. In the original publication it was clearly stated that this was not the case, as appears from the following quotations:

"The frictional resistance of sand to water within certain limits of size of grain and rate of flow varies directly as the rate and as the depth of sand. This is given by Piefke as Darcy's law. I have found that the friction also varies with the temperature, being twice as great

* Described in the Annual Report for 1892, p. 539.

Mr. Hazen. at the feezing point as at summer heat both for coarse and fine sands, and also that with different sands the resistance varies inversely as the square of the effective size of the sand grain."

The formula was then given for the flow of water in sands with effective sizes between 0.1 and 3.0 mm. It was then stated:

"For gravels with effective sizes above 3 millimeters the friction varies in such a way as to make the application of a general formula very difficult. As the size increases beyond this point, the velocity with a given head does not increase as rapidly as the square of the effective size; and with coarse gravels the velocity varies as the square root of the head instead of directly with the head as in sands. The influence of temperature also becomes less marked with the coarse gravels."

The table was therefore prepared showing the experimental results for gravels, not covered by the sand formula. It is also to be noted that in the original publication results were uniformly expressed in terms of meters daily in a solid column of water of the same area as that of the sand. In quoting the results in the present case the terms of the statement have been changed by assuming 40% of voids and by multiplying the figures by 2.5, besides otherwise changing the units of the formula.

The Lawrence formula may be relied on at this time, not alone or principally because of the relatively limited experience on which it was originally based, but because it has been widely applied since that time, and when properly applied has proved reliable.

It certainly does not discredit the formula that the coefficient varies with the character and cleanliness of the sand, and even to a certain extent with the "uniformity coefficient," which is a measure of the relative range in size of particles in the same sample. It is well known that there is a range in the coefficients to be used in computing the flow of water through pipes, but the fact that a higher coefficient is used for a clean and smooth pipe than for an old and dirty one is no reason for discarding the formula. In the flow of water through the much more complex passageways between the grains of sand there will obviously be variations in the coefficient, depending on the shape and smoothness of the particles, the way in which they are packed, and the quantity of finer foreign material also contained in those pores. As a matter of fact, the variation of c in the original formula,

$$V = c\, d^2 \frac{h}{l} (.70 + .03\, t),$$

is not especially great. The coefficient rarely falls below 400, even for old and dirty sand, and rarely rises above 1 200, and, in a majority of ordinary sands, falls between 700 and 1 000. This is certainly not a wide range.

It is to be noted that the formula was never intended to apply to clays, hardpans, soils, and other materials. The effort to apply it to

such materials is not to be encouraged, and the results are not to be depended on. Mr. Hazen.

The procedure of sifting a sample of the sand through rated sieves, plotting the results, and determining the effective size and the uniformity coefficient, is a very definite one, and comparatively simple. Nevertheless, erroneous results have frequently been reached, by failure to observe the fundamental requirements.

It may not be amiss at this time to define the procedure as to some of the matters regarding which errors have been made most frequently.

The effective size is that size such that 10% by weight of the particles are smaller than it. The decision to use 10% was reached by a study of experimental results with various sands at the Lawrence Experiment Station. It was found that the 10% line was a distinctly better basis for arranging the experimental results than either the 8% line or the 12% line, and therefore it was selected. The data on which this decision was reached were good, although not numerous; subsequent experience, with a greater variety of sands, has not shown reason for reconsidering the decision first reached.

The diameter of a sand grain is defined as that of a sphere of equal volume. This must be rigorously insisted on. Taking the diameter of grains as equal to the measured mesh of sieve or some such arbitary basis invariably leads to errors which may be large.

The method of rating the sieves described in the original publication was simple and sufficient. It consisted in shaking sieves with a sample of representative material precisely as in the ordinary course of analysis, then taking the sieves apart, giving each sieve separately a slight further amount of shaking, when the particles passing are all of nearly the same size, which size is larger than nearly all the particles that have passed and is smaller than nearly all the particles that remain on the sieve. Some of the particles resulting from this further shaking are weighed and counted, and the average diameter is calculated from the average weight; the size thus found is the size of separation of the sieve. This process is repeated several times, with representative sands of different types, and the average taken. The shaking is not to be continued until no more particles will pass, as often stated, but only until the bulk of that sand that will pass that sieve with a reasonable amount of shaking has passed, as, for example, to the point where doubling the number of shakes will not make considerable difference. There are always some holes larger than the average, and if shaking were continued until no more particles would pass, the process would be a very slow one, and the result would be to bring the size of separation up to the size of the largest holes.

The sieves must be of regularly woven wire cloth, and must be carefully rated. It has not been found possible to buy ready-made sieves which can be relied on without rating.

Mr. Hazen. The process of mechanical sand analysis, as a basis for estimating the flow of water in sands and gravels, is emphatically not one that can be taken up in a spare hour, in a way to yield reliable results; but, a careful manipulator, by following a perfectly simple procedure, may reach results which are trustworthy, if not precise, and which, in the words of the original publication, enable him to:

"* * * decide with confidence many otherwise indefinite points, and thus avoid unnecessary expense and unsatisfactory results from the use of unsuitable or poorly arranged materials."

As an illustration of the practical use of this formula, take the case of the filter construction now under way at Toronto, Ont. A large permanent plant is being built on a sand island, cut by channels, with foundation some feet below lake level. It was proposed to dig a drainage canal entirely around the site and pump the water out of it, thereby draining the whole site. Calculation made in August, 1908, assumed this drainage canal to be 3 500 ft. long, and that the depth of material under the canal carrying water would be 50 ft., making a total area of 4 acres through which the water would flow. The sand was assumed to have an average effective size of 0.25 mm. and c was taken as 800. It was assumed that the average distance from the canal to the water outside would be 200 ft., and that the water in the canal would be 5 ft. below lake level, resulting in a ground-water slope to the canal of 0.025. At a temperature of 50° water would pass, by the formula, at the rate of 1.25 m. per day, or 1 110 000 Imp. gal. per acre daily, and for the 4 acres of assumed cross-section of flow the total quantity of water would be 4 440 000 Imp. gal. per day. The writer does not think that any of the practical men who looked at the site before it was drained believed that the removal of so small a quantity of water would suffice. As a matter of fact, the actual quantity of water pumped has never exceeded the computed quantity, and has ordinarily been only one-half or one-third as much, owing in part to the fact that the sand proved to be finer than was assumed (actually about 0.21 mm. on an average), and that the average distance from the canal to open water was greater and the hydraulic slope less steep than assumed.

The use of the formula in this case allowed the installation of suitable equipment at the start, avoided false moves, and allowed the structures to be built economically on a site which, in the absence of such calculation, might have been assumed to be unsuitable because of the difficulty of handling the water in the sand.

Mr. Beardsley. R. C. Beardsley, Esq. (by letter).—This subject is of the greatest interest to the writer, and though some of the argument seems to be faulty, the paper should be of value in direct proportion to the amount of discussion it brings out.

The writer has built some twenty dams on soft bottoms, about

Mr. Beardsley.

half of them being on sand. A number of the earlier dams met with accidents, and each failure taught a lesson. Mr. Koenig's statement that textbooks emphatically discourage the construction of dams on sand bottoms is quite inaccurate, as not a single textbook is recalled that does this. "Hydro-Electric Plants," by the writer, devotes considerable space to the subject. The high cost of such dams, when properly built, may amount to "emphatic discouragement" to many, for it is true that for sand bottoms the cost of foundations is much the largest portion of the total cost. The engineer is usually compelled to dispense with precautions which he may think quite necessary for the safety of the dam, in order to reduce the cost to some figure guessed at by the owner or obtained from a source unknown even to himself.

In the writer's experience, the failures above mentioned have in every case been due to inadequate aprons or extension mats. The ideal dam for sand bottoms is one consisting of three essential parts: (1) At the up-stream edge there must be a water-tight row of steel sheet-piling, driven to proper depth, and fastened to the edge of the dam in a water-tight manner. (2) The dam, proper, must be of permanent and water-tight materials, and of such section that the pressures are uniformly distributed over the entire base. (3) Extending along the down-stream side of the dam, some means must be provided to conduct the water which passes over the dam's crest in such a manner that no serious vibrations will be set up, no vacuums, and so that the undermining of the dam will be prevented.

Fig. 8 shows diagrammatically the ideal dam, the rectangle, *A B C D,* merely represents the uniformly distributed, effective weight acting on the sand bottom and tending to resist the tipping and sliding of the structure.

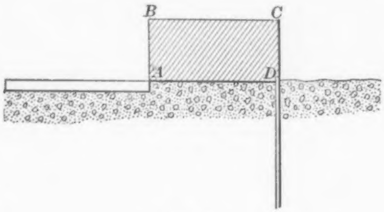

FIG. 8.

The writer knows of only two types of dam which give the same pressure on every square foot of the foundation—the gravity dam with apron, and the reversed dam.

In Fig. 4 Mr. Koenig shows a gravity dam, and calls it an Ambursen dam. The gravity dam was patented by the writer's father, E. R. Beardsley, many years before the Ambursen Company went into the dam business. It was described by him in a pamphlet entitled "The Gravity Dam," published in 1895. Mr. Beardsley not only explained the entire theory of such dams, but gave to the dam its name. The reversed dam is a new type, lately invented by the writer.*

* Described in *Engineering Record*, November 26th, 1910; and *Engineering Contracting*, February 1st, 1911.

Mr. Beardsley.

Either of these dams gives the desired distribution of pressure. All other dams have an excessive pressure at the down-stream edge, which, on sand bottoms, of course, is a disadvantage, such foundations having a questionable safe bearing value. Excessive pressures are especially to be avoided where there is the slightest danger of undermining.

According to the writer's experience, an ideal dam for the most treacherous bottoms is the reversed dam, shown in Fig. 9. This uses the gravity principle, but, instead of carrying the water load on the deck, it carries a much larger (two to three times larger) water load on the base. The dam, as shown, is filled in above with sand, although for the usual soft bottom this would not be required. Such filling will give the maximum degree of safety, as regards tipping, sliding, and seepage. The apron of the dam is also provided with an unusual

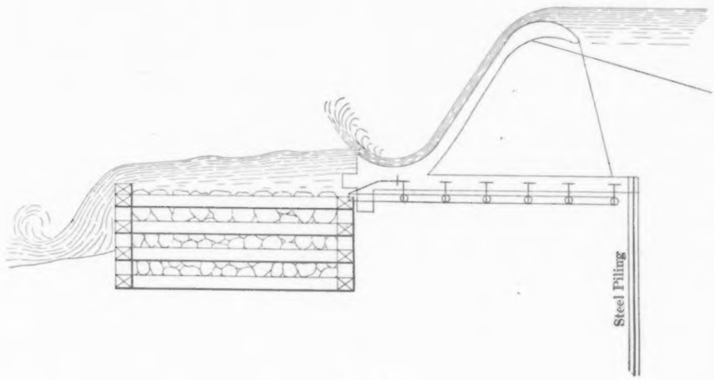

FIG. 9.

upward curve so as to aid in killing the velocity of the water leaving the dam. In the case of high dams, the lower portion of the apron would be covered with metal. The effect of such an apron should be similar to that of the deflector used for breaking up the discharge from high-head impulse wheels. The water falling from this curve would strike on the top of the cribbing, and escape, partly over the top and partly between the down-stream timbers, thus spreading the current and preventing the swirl which has proved fatal to most sand bottom dams. As the finest western sand will be eroded by a velocity of 1 ft. per sec., or less, and as it is a well-known fact that a swirl set up by water passing 4 ft. deep over a comparatively low dam will wash holes below the dam to a depth of 20 ft. or more, the importance of damping out all possible current will be appreciated. A crib mat is shown as an ideal. This type has many advantages, as it may settle

Mr. Beardsley.

without injury, is comparatively cheap, and has sufficient depth to distribute the escaping water. This mat must be anchored securely by chains or bolts, so that it cannot settle away from the dam.

In Mr. Koenig's paper too much importance has been given to up-lift under the dam, due to seepage. While every engineer should know that, unless such up-lift is eliminated, a failure is quite sure to follow, yet the elimination of such pressure is so easily and surely accomplished that it is not necessary to know how to calculate its intensity when allowed to exist. On the bottom of thousands of filter beds large quantities of water are collected through drains and conducted away from the beds without carrying away the sand. In the same way, the very small seepage under steel sheet-piling can be collected under the base of the dam and conducted safely to the water below.

The writer questions the existence of any considerable up-lift where only one row of piling is driven, and that along the up-stream edge, even though there is no resort to drainage; at least, he has never observed a pressure of more than 1 lb. per sq. in. under such conditions, even when wood piling was used. In all such river beds there is a constant underground flow, and part of the seepage under the piling will not come to the surface at all, but will continue on down the stream as ground-water. However, drainage costs so little that it should be installed in every case.

In the writer's opinion, the dam shown by Mr. Koenig in Fig. 1 is sure to fail unless the down-stream toe is so well reinforced that it cannot break. Two dams, built like that shown, failed. The pressures are very high at the toe—the point where the slope of the apron intersects the base—and, as indicated in Fig. 1, this point will become the tipping edge, unless reinforcement transfers the fulcrum to the down-stream edge of the mat. It would be necessary to have the top edge of the mat, at the down-stream edge, at a sufficient depth to reduce the velocity of the water escaping from the pocket to about 1 or 2 ft. per sec., otherwise the back-wash would undermine the mat (if as shown in that cut) in a short time.

Next to undermining below the dam, the greatest danger, the writer thinks, is that due to the horizontal down-stream thrust causing the entire dam to slide on the bed of sand.

Fig. 10 illustrates the very unstable condition of a dam resting on a bed of sand and with a deep hole washed out below it. In this case, the tendency is for the dam, and the entire mass of sand, *A B C D*, to slide down stream under the pressure represented by the pressure diagram, *D E F G*. The coefficient of friction along *A D* may be assumed as about 0.20 to 0.25, and the weight of materials must be calculated on the assumption that practically the entire structure and foundation is submerged and therefore has lost in weight 62.5 lb. per cu. ft.

Mr. Beardsley.

There is also the angle of repose of the sand, as *D A C,* which enters into the problem. This angle may be as small as 3° for gumbo, or from 10° to 15° for western sand. It will also be noted that the water is anything but calm along the surface *A B,* and that, in many dams, the overfalling water sets up a rhythmic vibration similar to pressure waves in a long penstock, and thus greatly reduces the coefficient of friction and the angle of repose.

As to the general proportions: The writer has never known of the water breaking under a gravity dam mat from the up-stream side when a row of sheet-piling was driven along the up-stream edge. Of the three cases where the dam was completely washed out, it is positively known that two were undermined by the back-wash along the down-stream side, and the third was pushed out, as explained in connection with Fig. 10. The latter dam was on the Elkhorn River, in Nebraska, and the foundation consisted of the finest sand mixed with silt. In all cases the dams were provided with an extension mat, from 16 to 24 ft. long, and yet, in a number of instances, the back-scouring

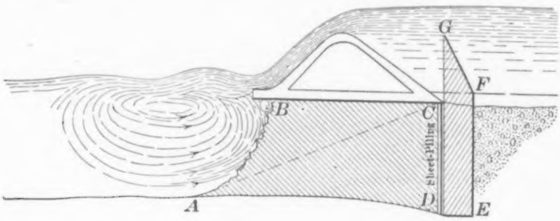

Fig. 10.

extended for a distance of from 40 to 60 ft. back under the dam. Had the extension mat been built so that it could settle, these accidents could not have happened. Piling should not be used for extension mats, and very seldom to support the dam, as such support takes the weight off the sand, and encourages a concentration of the seepage water. Wood sheet-piling should never be used unless the soil is free from large rock, and, in all cases, piles should be jetted down.

Crib mats of 10 by 10-in. timbers, filled with stone and sunk from 8 to 10 ft. in ordinary soil, cost about 15 cents per cu. ft., and, for a dam 20 ft. high, would cost from \$50 to \$60 per lin. ft. of dam, or considerably more than a complete 20-ft. dam would cost if built on rock bottom. To this cost must be added that of the sheet-piling. It has been the writer's custom to use piling at least 12 ft. long, and from that up to a length equal to the height of the dam. However, this practice has not been based on tests or actual knowledge of what the proper depth should be. By reference to Fig. 9, it will be noted that shorter piling would be required for a reversed dam than for any

Mr. Beardsley.

other, if the interior is filled with soil as indicated; for, in this case, a maximum length of seepage path to the top of the piling is secured without involving a flat surface, which might tend to cause the water to follow from the top surface of the fill down to the surface of the piling. Seepage will follow such surfaces for great distances, as in the case of pipes laid through earth dams, etc.

The greatest caution must be observed in keeping the foundation pressures well within safe limits. Thus, for sand bottoms mixed with more or less silt, Ira O. Baker, M. Am. Soc. C. E., states that the bearing value is practically zero. Such soil must be excavated, and a stratum must be reached which will support with safety the maximum pressures. A reversed dam 100 ft. high has a maximum founda-

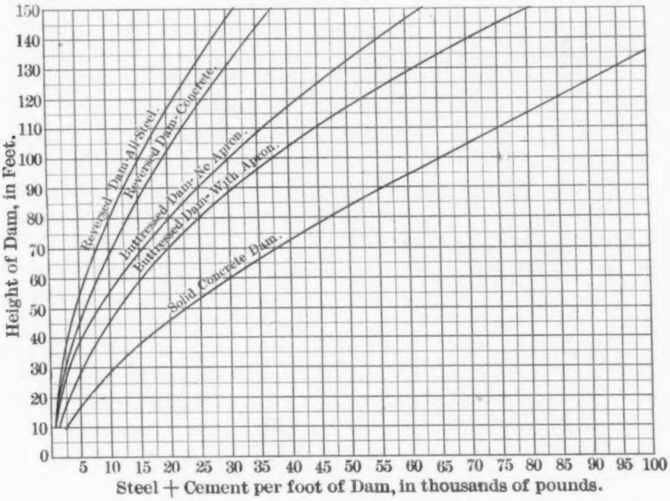

Fig. 11.

tion pressure of 5 000 lb. per sq. ft., or less, while the solid masonry dam, as usually built, has pressures exceeding 20 000 lb. per sq. ft.

Many failures are caused by inadequate foundations. Considerable time is required to make proper soundings, and the cost is a large item, hence, most dams are built by guess. The surface of the river bed is taken as a fair sample of the soil at greater depths, while, as a matter of fact, it is difficult to find a foundation which does not present much the best appearance on the surface.

The reversed dam is the cheapest of all permanent dams, and also the safest. Fig. 11 indicates the great saving in materials effected by it, and gives the relative weight of transported materials, steel + cement, for various types.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

LEFFERT LEFFERTS BUCK, M. Am. Soc. C. E.*

DIED JULY 17TH, 1909.

Leffert Lefferts Buck, third son of Lemuel and Elizabeth (Baldridge) Buck, was born on February 5th, 1837, in Canton, St. Lawrence Co., N. Y. He was descended from sturdy New England stock, his grandfather, some of whose immediate family attained considerable prominence in the Revolutionary War, having moved from Connecticut early in the 19th Century.

As a boy, Mr. Buck evinced that keen ambition to accomplish extraordinary results which in after life so strongly characterized him, and, in his home and school duties, developed the ingenuity and manual efficiency, which ultimately became remarkable. He then, as always, eagerly sought opportunity to cope with difficulties and to excel in feats of mind, skill, or strength.

He entered the first class of the college which afterward became St. Lawrence University, but had hardly completed the college course when the Civil War broke out, and he at once determined to enter the Army. His father, who was a close friend of the Governor of New York, wished to secure a commission for him, which he could doubtless have done; but Mr. Buck, with that characteristic regard for fitness which he applied to himself even more rigidly than to others, refused it because of his lack of military training, and in July, 1861, enlisted as a private in Company A, 60th New York Infantry. This was one of the most serious mistakes of his life, for, despite his unwillingness to accept the fact without test, he possessed an innate military ability that far outweighed his lack of previous training, and would doubtless have attained high rank had he entered the Army as a commissioned officer instead of as a private. However, he rose through the ranks of Corporal, Sergeant, Lieutenant, and Captain, until mustered out at the close of the war with the brevet rank of Major. He participated in some of the most arduous campaigns of the war, including the battles of Antietam, Chancellorsville, Gettysburg, Lookout Mountain, Ringgold, Resaca, Kennesaw Mountain, Peachtree Creek, and Atlanta, and also in Sherman's March to the Sea. In the famous "Fight Above the Clouds" at Lookout Mountain, it was he who finally carried the Union Flag to the summit, after two color-bearers had been shot down under it.

His indomitable will and contemptuous disregard for the restraining

* Memoir prepared by R. S. Buck, M. Am. Soc. C. E.

power of wounds and disease, were shown at Antietam where, after being shot through both legs, he kept afoot with his command for the rest of the day, and only after a night on the cold ground and the hopeless stiffening of his wounded legs made further walking impossible, could he be forced into the hospital. At another time, when recovering from a severe attack of typhoid, and still hardly able to walk, he learned that an important engagement was imminent. The surgeon refused to release him from the hospital, and, to aid in securing observance of his orders, put his patient's uniform out of his reach. However, Mr. Buck managed somehow to get his sword, and in his hospital clothes showed up with his command. He was again wounded at Resaca.

While Mr. Buck liked to talk over events of the war, he rarely spoke of those incidents wherein his personal participation was entitled to distinction. He always warmly esteemed and liked to meet those who faced him on the other side, and often expressed the wish that the soldiers of the North and South might some day fight side by side against a common enemy. While deploring many of the results of war he often said that he thought war preferable to too much peace, as he felt that too much peace bred indolence, selfishness, and avarice, while war developed the nobler traits of men.

While favoring an intelligent and fair pension system, Mr. Buck always greatly deplored what he considered a tendency to capitalize patriotism into an indiscriminate distribution of pensions for political purposes, and always refused to consider a pension for himself, although his wounds and health fully entitled him to it under the law.

He earnestly desired to remain in the regular Army at the close of the war, but decided not to do so because he lacked West Point training and was too old to enter the Military Academy. Immediately on being mustered out, he entered the Rensselaer Polytechnic Institute of Troy, N. Y., where he took the prescribed four years' course in Civil Engineering in three years, and was graduated in 1868.

For about three years after graduating, he served as Assistant Engineer in the Croton Aqueduct Department of New York City. He then spent two years in Peru, where he erected the first Verrugas Viaduct on the Lima and Oroya Railroad, and assisted in other work of construction on that road. The first Verrugas Viaduct was destroyed by flood some years later, and he designed and planned the erection of another, which is still in use, although most of the bridges on the same line have been replaced to meet the growing demands of traffic.

On his return from Peru, in 1873, Mr. Buck supervised the manufacture of the material for the bridge at Louisiana, Mo., one of the first large railroad bridges across the Mississippi River.

In 1875 he returned to Peru for about two years in charge of bridge erection.

In 1877 he became connected with the Railroad Suspension Bridge at Niagara Falls, which had been built in 1850 by the late John A. Roebling, M. Am. Soc. C. E. He reinforced the anchorages of the bridge, which had been badly overstrained, and replaced a large number of cable wires which had become seriously corroded about the anchorage shoes.

In 1880 he replaced the wooden suspended superstructure with one of steel, and in 1885 he replaced with steel towers the original stone towers, which were deteriorating under the combined influence of frost and motion due to moving loads. All this work of removal was accomplished without interruption of traffic, and was the most difficult, delicate, and daring piece of bridgework ever undertaken. This statement is made with full appreciation of the many great accomplishments since that time, but also with appreciation of the lack of precedent to guide and the complete dependence on his own ingenuity that faced the engineer in that work. Several friends, whose opinions were entitled to very serious consideration, urged him to suspend travel while doing this work, but he never once wavered in his determination to carry out the work as he had planned it.

In 1881, the Norman Medal of the American Society of Civil Engineers was awarded him for his paper on "The Reinforcement of the Anchorage and Renewal of the Suspended Superstructure of the Niagara Railroad Suspension Bridge."*

As finally reconstructed, the Niagara Railway Suspension Bridge served until 1896, when it was replaced by the present massive, two-hinged, spandrel braced, steel arch, which was designed by, manufactured, and erected under the direction of Mr. Buck. This arch was erected on the same center line as the Suspension Bridge, and the latter was taken down without interruption of traffic.

Associated with G. W. McNulty, M. Am. Soc. C. E., in 1888-89, Mr. Buck rebuilt the Niagara Falls and Clifton Bridge, a highway suspension bridge just below the Falls, which was originally built by Mr. James Keefer, in 1850. In 1897, in order to provide capacity for trolley cars as well as for the increasing crowds of visitors, the Niagara Falls and Clifton Suspension Bridge was replaced by the longest steel arch span in the world, which was designed by and constructed under the direction of Mr. Buck. This work required maintenance of traffic during construction; however, in order to save time, expense, and risk, traffic was suspended for a few days when the arch was closed.

For his paper on this bridge, the Institution of Civil Engineers awarded him the Telford Premium in 1901.

Mr. Buck was Consulting Engineer of the Lewiston and Queenston Suspension Bridge across the Niagara River, built in 1899. He designed and constructed the Driving Park Avenue Bridge, and the

* *Transactions*, Am. Soc. C. E., Vol. X. p. 195.

Platte Street Bridge, in Rochester, N. Y. The former was the first spandrel braced arch built in the United States. He also designed and erected a number of bridges on the Northern Pacific Railroad during the extension of that road to the Pacific Coast.

By invitation of the Government Engineers, he made designs for the Connecticut Avenue Bridge and the Memorial Bridge across the Potomac at Washington.

His greatest work, at least in point of magnitude, is the Williamsburg Bridge across the East River between New York and Brooklyn. He was Chief Engineer of this bridge from the time of its inception in 1895 until its completion in 1903. After this he was retained as Consulting Engineer in the Department of Bridges in connection with this and other bridges, until a short time before his death, when he resigned on account of failing health.

For several years he represented Peru and Ecuador on the International Railway Commission, which planned a series of railroads connecting the United States with South America.

On June 4th, 1902, Mr. Buck married Miss Mira Rebecca Gould, of Paducah, Ky., who survives him.

Leffert Lefferts Buck will always stand forth as one of the heroic figures in the history of Engineering. He was resourceful, self-reliant, bold and aggressive in action, but as little disposed to force his ideas upon others as he was to let others force their ideas upon him, or force him from his own. He reached conclusions promptly and decisively, depended firmly on his own judgment, and was often resentful of divergent outside influences and opinions. He brought into his work a practical mechanical knowledge and a singleness of purpose which, while they sometimes restricted his vision of controlling necessities and ultimate broader aims, always placed on his work the stamp of originality and ability, often of a genius striding well ahead of his time.

His personal character was striking, and compelled admiration. In him the Puritan strain was strongly developed. Spartan-like, rugged, uncompromising always, and often showing a tenacity in the face of what to others appeared overwhelmingly adverse proofs, there was in his thoughts and actions a simple, earnest honesty, and an indomitable and unselfish love of right as he saw it. Below his severe, sometimes forbidding, exterior, was a loyalty, a generosity, a tenderness almost feminine.

Few men have steered a more direct course in life, wavered less in their estimates of right and wrong, or made less effort to win friendship and applause; and yet few men have had a greater number of devoted and loyal friends, or received more spontaneous and generous applause.

Mr. Buck was a Member of the Institution of Civil Engineers of England, the Loyal Legion of the United States, the Century Associa-

tion, the Engineers Club of New York, the Hamilton Club of Brooklyn, the Army and Navy Club, and the St. Lawrence County Society.

He was elected a Member of the American Society of Civil Engineers on February 3d, 1875, and served as a Director from 1892 to 1894.

FRANK LESLIE DAVIS, M. AM. Soc. C. E.*

DIED JUNE 9TH, 1909.

Frank Leslie Davis, son of David F. and Louisa Davis, was born at Wellsville, Ohio, on December 2d, 1858, and was reared on a farm. He taught school in Warren County, Iowa, and in 1884 was graduated from Simpson College. Later, he spent one school year taking special engineering work at Kansas State University.

Mr. Davis commenced his professional career with the Chicago, Rock Island and Pacific Railroad in 1887, at Wichita, Kans., where he had charge of forty miles of new construction work. He devoted the following ten years almost entirely to railroad engineering, during which time he was employed in various capacities on construction and location work by the Chicago, Rock Island and Pacific, the Southern Pacific, the Northern Pacific, and the Great Northern Railroads.

In 1897-98 Mr. Davis was City Engineer of Tacoma, Wash. In 1903-04 he was Assistant Engineer on the construction of the Puget Sound Power Company's plant near Tacoma. In 1904 he was again elected City Engineer of Tacoma, and continued to occupy this position until his death.

On October 14th, 1891, Mr. Davis was married, at Knoxville, Iowa, to Miss Alice Scoles, daughter of the Rev. J. J. D. Scoles, with whom he had become acquainted while at Simpson College. Mrs. Davis and a daughter, Miss Leslie S. Davis, survive him.

Mr. Davis was a prominent member of the Masonic Fraternity, holding the 32d degree. He was also a Shriner, a member of the Patrol of Afifi, and Master of A. F. & A. M. No. 22, of Tacoma, Wash.

He was a man of most exemplary habits, of strong convictions, and of aggressive initiative. As an engineer Mr. Davis contributed his full share to the upbuilding of the West, and, as a citizen, to the welfare of the community in which he spent his later years. The affectionate esteem in which he was held by his friends, by business and fraternal associates, and by his fellow-citizens generally, was most strikingly shown in the wide manifestation of sorrow and of sympathy for his family on the occasion of his death.

Mr. Davis was elected a Member of the American Society of Civil Engineers on September 6th, 1905.

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* Memoir prepared by Arthur L. Adams, M. Am. Soc. C. E.

JOSEPH PALMER FRIZELL, M. Am. Soc. C. E.*

Died May 4th, 1910.

Joseph Palmer Frizell was born at Barford, Quebec, Canada, on March 13th, 1832. His parents, Oliver and Mary Beach Frizell, were natives of Vermont and belonged to good New England stock. His early education was received at Brownington, Vt., and Richmond, Canada. His teachers, discovering his aptitude in mathematics, taught him algebra and geometry, advising that he fit himself for Civil Engineering.

At the age of eighteen Mr. Frizell went to Manchester, N. H., where he was employed in a cotton mill until 1854, when he entered the City Engineer's office as an assistant. He devoted all his leisure time, both day and evening, to the study of his profession, and remained with the City Engineer until the close of 1856.

In 1857 he entered the office of the Locks and Canals Company, controlling the water power on the Merrimac River at Lowell, Mass., under the charge of the late James B. Francis, Past-President, Am. Soc. C. E., the eminent hydraulic engineer. Mr. Frizell was Assistant Engineer until 1861, when he left on account of the Civil War. He returned to this work in 1866, and remained until 1867. While in the Locks and Canals office he assisted Mr. Francis in the preparation of his treatise entitled "Lowell Hydraulic Experiments." In the preface of the edition of 1868, the author acknowledges his obligation to various persons, and also "to Mr. Joseph P. Frizell, now of Davenport, Iowa, to whom he is indebted for assistance in some points involving the higher mathematics."

In 1861, on the breaking out of the Civil War, he entered the United States Government Service as Assistant Civil Engineer, and was engaged until 1865 on the construction of Government fortifications along the Gulf Coast. He was stationed at Ship Island for 3 years, and at Davenport, Iowa, for 2½ years.

In 1870 Mr. Frizell opened an office for general consulting practice in Boston, where he remained until 1878. He made many hydraulic investigations, including reports on the Minneapolis water-power in 1875, and on damages from diversions on the Charles River, Waltham, and on the Neponset River and Mother Brook. In 1878 he went West as Assistant Civil Engineer to the U. S. Engineering Department, and was engaged in various hydraulic investigations, among which were the construction of reservoirs at the head-waters of the Mississippi River, investigations at the Falls of St. Anthony, and other similar matters. In 1881 he served on the Water Commission at St. Paul,

* Memoir prepared by R. A. Hale, M. Am. Soc. C. E.

reporting on the water supply. During his work throughout the West, in the U. S. Engineering Department, the letters received by him from various army officers showed the high esteem in which he was held.

In 1890-92 Mr. Frizell was Chief Engineer of the Board of Public Works at Austin, Tex., during which time he made a design for, and commenced the construction of, the Austin Dam. On account of various restrictions by the Board of Public Works, he resigned this position in June, 1892, after which the dam was completed upon a somewhat modified plan. The general history of this work is stated in various engineering publications.

In 1893 he resumed his Boston office and practised until 1903, when he retired from active business. During this last period in Boston Mr. Frizell was connected with various water-diversion cases on the Nashua River due to the taking by the Metropolitan Water Board.

In 1878 he obtained patents for compressing air by hydraulic methods, using inverted siphons and air chambers, etc., and also for pumping water by the use of compressed air, both of which have found practical application. The first method is described in detail in a paper by Mr. William Webber, entitled "Water Power by Direct Air Compression."* Air compressor plants built on these general principles have been installed at Norwich, Conn., Magog, Canada, and are in use in European countries. The Pohle air-lift pump is an illustration of the application of other methods.

In 1900 Mr. Frizell published a book on "Water Power" which was an interesting addition to hydraulic literature, and he was also a frequent contributor to various scientific publications. Among them is an article on the "Strains in Continuous Beams."† He was collaborator on Johnson's Civil Engineering Cyclopedia. Among the papers contributed to the *Transactions* of the American Society of Civil Engineers may be mentioned "The Water-Power of the Falls of St. Anthony," Vol. XII; "The Storage and Pondage of Water," Vol. XXXI; and "The Old Time Water Wheels of America," Vol. XXVIII. He was interested in turbine water-wheels, and made a study in developing a formula by which the discharge of a turbine of the Boyden type could be computed from the various angles of the guides and buckets. He was a remarkable mathematician, and was self-taught in the higher mathematics.

His son, Dr. Arthur B. Frizell, writes as follows:

"My father's professional reputation was essentially scientific; he was valued for his ability to carry through a piece of work requiring original investigation. His success in dealing with difficult problems was owing to mathematical aptitudes nearly akin to genius.

* *Journal*, Association of Engineering Societies, Vol. 26, p. 35.

† *Journal*, Franklin Institute, 1872.

With the faculty of formulating engineering questions mathematically he combined rare skill in the use of the delicate analysis by aid of infinitesimals. To-day such faculty is generally the result of protracted academic training in methods more accessible to the average mind; but my father was self-taught, at least in calculus; his processes were intuitive and sure.

"There is an interesting entry in his diary to the effect that the ideas on which calculus is based admit of much wider application, indeed, are capable of guiding us in every study that proceeds by exact reasoning. Fifty years later, Professor Schoenflies of Königsberg, in a report on the progress on the newest branch of mathematics, which is largely concerned with precisely these underlying principles of calculus, asserted that its development from a set of axioms, analogous to those of Euclid, would be equivalent to an analysis of the foundations of all human knowledge."

From 1903 until his death in 1910, Mr. Frizell resided in Dorchester, Mass. He was a member of the Tariff Reform League, and took an active interest in writing papers on the subject. He was a member of the Unitarian Church, which he attended regularly with his family. Although of a quiet and somewhat reserved disposition, he had many warm friends, and the personal letters of Hon. George F. Hoar, George S. Shattuck, Esq., and other prominent men, testify to their appreciation of his ability and friendship.

On October 17th, 1864, Mr. Frizell was married to Julia A. Bowes, of Boston, Mass. He is survived by Mrs. Frizell and two children, Dr. Arthur B. Frizell, a member of the faculty of Kansas State University, and Miss Alice M. Frizell, who resides with her mother in Cambridge, Mass.

Mr. Frizell was elected a Member of the American Society of Civil Engineers on January 3d, 1883.

HENRY RANDOLPH HOLBROOK, M. Am. Soc. C. E.*

DIED DECEMBER 21ST, 1907.

Henry Randolph Holbrook, son of Nathan K. and Jane (Porter) Holbrook, was born on November 22d, 1838, at Columbia, Conn. He was educated in the common schools, and at the age of 17 left home to make his way in the world.

From February, 1865, to September, 1866, Mr. Holbrook was engaged as Assistant Engineer in charge of construction on the Missouri Pacific Railroad. In October, 1866, he joined the Engineer Corps of the Eastern Division of the Union Pacific Railroad, remaining with the Corps until August, 1868.

* Memoir prepared by Augustine W. Wright, M. Am. Soc. C. E.

In April, 1867, an Engineer Corps, in charge of the late William H. Greenwood, M. Am. Soc. C. E., as Chief Engineer, Mr. Holbrook being his Principal Assistant Engineer, left Junction City, Kans., to make a survey of the line to Denver, Colo. After locating the line between Forts Hays and Elsworth and continuing the preliminary surveys to Denver, the Union Pacific Railroad Company decided to make a survey through to San Diego, Cal. Five parties were sent out, two on the 32d Parallel and three on the 35th Parallel, Mr. Holbrook having charge of one of the latter parties.

From August, 1868, to September, 1869, he had charge of the location of the St. Louis and San Francisco Railroad from Jerome to Springfield, Mo., a distance of 100 miles, and from September, 1869, to September, 1870, he was employed in locating and had charge of the construction of 70 miles on the Kansas Branch of the Union Pacific Railroad, from Kit Carson, Colo., westward.

Mr. Holbrook was Chief Engineer of the Denver and Boulder Valley Railroad during its construction, from September, 1870, to February, 1871, after which he was in charge of a locating party on the Denver and Rio Grande Railroad. In March, 1872, he went to Mexico, where he remained until July, making reconnaissances for railroads in connection with this same road.

From July, 1874, to December, 1875, he was employed as Chief Engineer on the construction of the Pueblo and Salt Lake Railroad, from Las Animas to Pueblo, Colo., and was Resident Engineer on the Atchison, Topeka, and Santa Fé Railroad from March, 1878, to July, 1880.

The Eastern Division of the Union Pacific Railroad was not built beyond Denver, Colo., the Southern Pacific ultimately using the 32d Parallel for its route and the Atchison, Topeka, and Santa Fé the 35th Parallel, and from July, 1880, to January, 1882, Mr. Holbrook was Chief Engineer of the Atlantic and Pacific portions of the latter line.

In 1882, he returned to Mexico, where he spent that year and 1883 in making examinations for proposed railroads.

In April, 1887, Mr. Holbrook was again appointed Chief Engineer of the Pueblo and Salt Lake Railroad, remaining in that position until 1890, from which time until his death, on December 21st, 1907, he devoted himself to irrigation and real estate projects, making his home at Pueblo, Colo., where he died. A brother and three sisters survive him.

Mr. Holbrook will always be remembered for "the best portion of a good man's life: his little, nameless, unremembered acts of kindness and of love."

He was elected a Member of the American Society of Civil Engineers on January 2d, 1890.

ALBERT VICTOR KELLOGG, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1911.

Albert Victor Kellogg was born at Aurora, Ill., on August 4th, 1863. He was the third son of the late Professor J. F. Kellogg, who occupied the head chair of Mathematics at Northwestern University, Evanston, Ill., where his son received his collegiate training.

Mr. Kellogg had given special attention to mathematics, and, having natural inclination and talent for engineering work, he began his career, in 1881, as a Rodman on location in the Engineering Department of the Chicago and Northwestern Railway. In 1884 he became Assistant Engineer of the Chicago and Evanston Railway, a part of the Chicago, Minneapolis, and St. Paul, and later was employed on surveys for the location of the Fremont, Elkhorn, and Missouri Valley Railway (now a part of the Chicago and Northwestern Lines), on which he worked as Assistant Engineer.

During the interim between this and subsequent railway work, Mr. Kellogg was engaged in municipal engineering in Omaha and neighboring cities in Nebraska, being associated with the late Andrew Rosewater, M. Am. Soc. C. E. He was employed principally as Assistant Engineer in charge of the surveys and construction of the sewerage systems of Grand Island, Hastings, and Plattsmouth, Nebr. Following this work, he was engaged in the general practice of engineering in St. Joseph, Mo,, and also in Denison and other cities in Texas.

About 1890, he re-entered railway service, and was connected with several Texas lines, notably the Missouri, Kansas, and Texas Railway, on which he held a responsible position in charge of construction and later in the Engineering Department of the operated line.

In 1893, when construction work was hindered by the financial panic, Mr. Kellogg became Engineer of the Right-of-Way Department of the Houston and Texas Central Railroad, at Houston, Tex., where he rendered valuable service in re-surveys, and in perfecting and systematizing records. He established himself so firmly in the confidence of the officers that he was put in charge of the next construction work undertaken by that Company. This work was done so thoroughly and so well that his ability was recognized, and, as a result, in 1901, he was put in charge of the Maintenance-of-Way Department of the Houston and Texas Central, and Houston East and West Texas Lines, subsidiary lines to the Southern Pacific Railway, Atlantic System. He continued in this position until 1904, when he was appointed Engineer of Maintenance of Way of the Sunset Lines, the Atlantic System of the Southern Pacific Railway, comprising the Morgan Louisiana and

* Memoir prepared by R. D. Parker, Assoc. M. Am. Soc. C. E.

Texas Railroad and Steamship Company, the Louisiana Western Railroad Company, Texas and New Orleans Railroad Company, and the Galveston, Harrisburg, and San Antonio Railway Company, with headquarters at Houston, Tex., which position he held until his death at Santa Rosa Hospital in San Antonio, Tex., on February 15th, 1911.

Mr. Kellogg possessed the qualities which go into the make-up of a big man. We who are left but the memory of this friend, appreciate the example he gave us in his energetic work, his fidelity to a trust and to a friend, and his loyalty to a service set apart for him. His life was that of a builder for the benefit of mankind, and his work was well done.

Mr. Kellogg was elected an Associate Member of the American Society of Civil Engineers on November 6th, 1895.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

CONTENTS

PLATES

Vol. XXXVII. MAY, 1911. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD. CERTAIN ENGINEERING STRUCTURES OF THE NEW YORK TERMINAL AREA.

By George B. Francis and Joseph H. O'Brien,
Members, Am. Soc. C. E.

To be Presented September 6th, 1911.

Introduction.

The location, general purpose, preparation of site, train schedules, trackage details, method of operation, and service work, have been described in other papers; this paper is confined to a description of the physical construction work of bridging over the approaches, bridging for the surrounding streets, foundations, substructures for various services, and the steel design for the Station Building.

The work herein described was placed under construction through the following primary contracts:

First.—A contract with the "American Bridge Company, of New York," dated June 1st, 1905, for the manufacture and delivery of the steel for street bridging, for the entire terminal, comprising about eight acres. The work under this contract was started in June, and the first deliveries were made in August, 1905. The major portion was delivered by January 31st, 1907, and all deliveries were completed

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

by April 30th, 1909. The work fabricated and delivered under this contract amounted to about 23 500 tons.

Second.—A contract with the "New York Contracting Company-Pennsylvania Terminal," dated June 21st, 1905, for the "Easterly Portion," embracing all excavation, masonry, and steel construction from the west line of Seventh Avenue, eastward to the normal tunnel sections in 32d and 33d Streets, covering about 290 ft. of line in 32d Street and about 500 ft. in 33d Street. The work was started on June 13th, 1906, and was practically completed in July, 1909. The amount of work done on the more important items was substantially as follows:

Excavation, earth	39 000 cu. yd.
Excavation, rock	133 794 cu. yd.
Concrete	28 930 cu. yd.
Water-proofing	17 500 sq. yd.
Back-filling, earth	58 850 cu. yd.
Duct work	47 676 duct ft.
Drainage	2 557 lin. ft.
Steel erected	8 944.8 tons.

Third.—A contract with the "New York Contracting Company-Pennsylvania Terminal," dated June 21st, 1905, for the construction of the viaducts west of Seventh Avenue, embracing all excavation, masonry, steel erection, and street-surfacing construction, for the viaducts for Eighth and Ninth Avenues, and 31st and 33d Streets. The work was started on October 19th, 1905, and was practically completed in January, 1910. The amount of work done on the more important items was substantially as follows:

Excavation, earth	2 300 cu. yd.
Excavation, rock	8 900 cu. yd.
Concrete	34 000 cu. yd.
Water-proofing	30 500 sq. yd.
Back-filling, earth	67 000 cu. yd.
Steel erected	14 400 tons.

Fourth.—A contract with the "New York Contracting Company-Pennsylvania Terminal," dated May 9th, 1906, for the substructures, embracing the excavation for and construction of all sub-

structures below track grade, such as foundations for the Passenger Station Building, pipe tunnels, baggage-trucking tunnels, electric ducts, drainage system, elevator pits, etc., etc. The work was started on June 1st, 1906, and was practically completed in January, 1910. The amount of work done on the more important items was substantially as follows:

Excavation, earth	8 300	cu. yd.
Excavation, rock	113 400	cu. yd.
Back-fill, earth	36 600	cu. yd.
Concrete	46 000	cu. yd.
Water-proofing	58 700	sq. yd.
Duct work	348 000	duct ft.
Drainage	33 700	lin. ft.

Fifth.—A contract with "Milliken Brothers, Incorporated," dated January 9th, 1906, for the manufacture and delivery of steel for the Passenger Station Building. The work was started immediately, and the first deliveries were made in May, 1907.

The contractors went into the hands of receivers in June, 1907, after delivering about 500 tons of material. By agreements with the receivers, dated September 6th and September 30th, 1907, this firm completed and delivered about 1 850 tons of additional material by November, 1907. The remaining steel for this building was furnished by other contractors.

Sixth.—A contract with "Milliken Brothers, Incorporated," dated January 9th, 1906, for the manufacture and delivery of steel for the Terminal Service Building. The work was started immediately, and the first deliveries were made in October, 1906. All deliveries were completed in April, 1907. The amount fabricated and delivered was about 2 437 tons.

Seventh.—A contract with the "American Bridge Company, of New York," dated July 17th, 1907, for the manufacture and delivery of steel for the Passenger Station Building, not furnished by Milliken Brothers or Receivers of Milliken Brothers. The work was started in September, 1907, and all deliveries were completed in March, 1909. The amount of work fabricated and delivered was about 24 662 tons.

Easterly Portion.

An important division of the terminal work was designated the "Easterly Portion." It embraced all the work of excavation, masonry, and bridging east of the west line of Seventh Avenue, and extending eastward under 32d and 33d Streets to the normal tunnel sections, covering an area of 1.88 acres, of which 0.51 acre is under private property and the remainder under public highways.

The franchise required that the tops of the finished structures should be 19 ft. below the surface of the streets and avenue. As this requirement was obviously made to provide space for a future rapid transit subway in Seventh Avenue, consideration was first given to the feasibility of effecting an arrangement with the city for the joint construction of the railroad's structure and a section of rapid transit subway in Seventh Avenue supported thereon.

Such an arrangement, however, could not be effected in any reasonable manner, owing to the impracticability at that time of determining future subway requirements, and therefore it was determined to restore the highways by back-filling over the railroad's structures. The portions of the latter under public thoroughfares were designed to support the load due to this heavy fill in addition to a uniform live load of 300 lb. per sq. ft., this total superimposed load aggregating about 3 500 lb. per sq. ft.

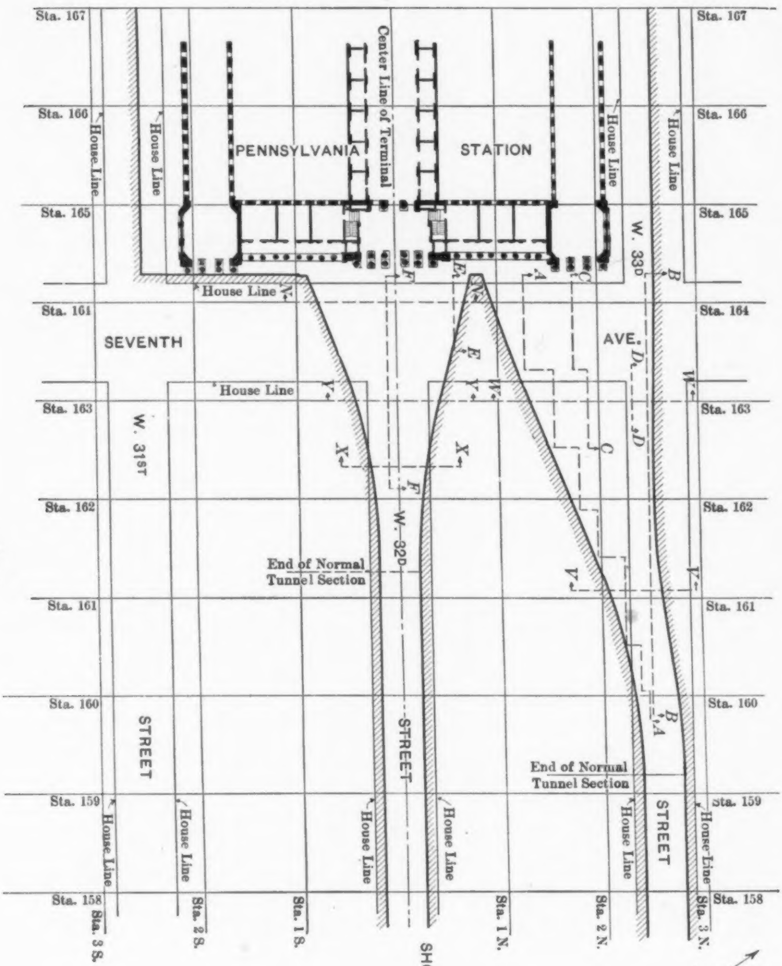
Owing to the great value of the private property affected, it was determined to design the structures beneath the private premises to sustain a superimposed load of 5 tons per sq. ft. The profile of the railroad line established the top of rail at about 49 ft. below the street level, and an under-clearance of 16 ft. 2 in. above the top of rail was also determined. Borings at the site disclosed the fact that rock underlaid the surface at an average depth of about 13 ft. A steel-roofed tunnel with masonry abutment walls and intermediate steel bents was adopted, and it was determined to set the steel roof girders parallel with the axis of Seventh Avenue in order to facilitate construction and provide properly for the maintenance of the avenue traffic.

Bents were introduced between tracks wherever practicable, but, owing to the track layout, long spans could not be wholly avoided. The spans vary from 20 ft. 6 in. to 81 ft. 2½ in. from center to center of bearings. The girders vary in depth from 3 to 9 ft., and are spaced

at 4-ft. centers where loads permit, but at 2-ft. centers where provision is made for loads from future buildings. To permit of simple details, and to avoid the use of excessively thick material, one-eighth of the deep girder webs was counted as flange section, and web splices were designed to resist moment. Cut granite templates are provided for the wall bearings of the smaller girders, and steel I-beam grillages for the wall bearings of the heavily loaded girders. These grillages are built up with separators, and provided with top and bottom countersunk riveted plates. The columns in intermediate bents are made up of two 15-in. channels, set about 15 in. from back to back, reinforced by 15-in. cover-plates, and double-latticed with $2\frac{1}{2}$ by $\frac{3}{8}$-in. bars, provided with riveted diaphragms at top and bottom, with riveted cap plates, field-connected plate, and angle knee-braces, and riveted plate and angle bases. The columns bear on cut granite templates capping concrete bases where loads permit, but the more heavily loaded columns bear on steel I-beam grillages, which in turn rest on granite masonry with concrete bases footed on the rock. Girders rest on the tops of columns. Curb angles are attached by riveted connections to the columns of all bents 4 ft. above top of rail for the protection of the fender walls.

The abutment walls are of gravity sections of 1:2:4 concrete, designed to carry the roof loads only, with an allowance of 30 tons per sq. ft. bearing value under the grillages. A granolithic top dressing is laid on top of these abutment walls under all grillage bearings. In situations where property was less valuable, and other conditions less restricting, the design of abutment walls capable of resisting hydrostatic pressure due to a head measured from the bottoms of the sewers might have been warranted. In this instance, however, it was determined to design the walls as above stated, trusting to a drainage scheme to relieve these walls of hydrostatic pressure.

The scheme consists merely of the application of well-known materials in a simple but unusual manner, namely, the space from standard section line to ledge, varying from 6 in. to 4 or 5 ft., is filled back of a suitable form to the full height of the abutment walls, with a porous concrete, 1 part cement, 4 parts sand, and 8 parts stone. This porous backing is faced with hollow, light-burned, porous, terra cotta, 12 by 12 by 4-in. partition blocks, laid with openings set vertically, and bonded for the continuity of the vertical hollow spaces. These



KEY DIAGRAM SHOWING LOCATION OF SECTIONS

Fig. 1.

blocks are anchored to the concrete backing by cut nails, driven into the latter and clinched over the inner walls of the blocks or joints. The bed joints are buttered in such manner that the back half of the joint is left open as far as practicable; the build joints are made in a similar manner.

The tile drainage sheet is footed on a longitudinal tile drain, made by chipping out the end walls and partitions of the blocks. This longitudinal drain is connected with a gutter at the toe of the abutment wall, which, in turn, is connected to the under-drainage system. On the face of the tiling above described, the wall water-proofing is placed for the full height, with dry laps extended over the backing wall and weighted down. A section at a time, varying from 25 to 50 ft., was thus prepared, and the abutment walls were completed as soon as practicable after water-proofing.

The roof cover, or so-called floor-plate, consists of plain I-beams, 4 and 6 in. deep, embedded in concrete and laid across the girders at intervals of 9 in. from center to center. The beams are not attached to the girders. The roof water-proofing is laid on top of the concrete-steel floor-plate, and a protecting cover of concrete is laid over the water-proofing; after this had been done the highways were back-filled.

The method of placing the concrete-steel floor-plate was as follows: First, forms were set between girders, then the plain I-beams were assembled, wire-brushed, spaced, and pinned up to grade; then mortar bearings were made for each beam over each girder, thus permitting, after the mortar had set, the removal of all wedges and spacers, and the cleaning of forms; after which the spaces beneath and between the beams were grouted, and the concrete was screeded to grade.

The protecting cover over the water-proofing was designed to be of plain concrete from 4 to 6 in. thick. As long as the back-filling was kept well back of the end of the completed work, and was stepped off in bench formation, the plain concrete cover served its purpose, but, in one case, when the back-filling was advanced in bank formation, close upon the completed construction work, the concrete cover broke, and the water-proofing was damaged, requiring the removal of much back-filling to effect proper repairs. After the occurrence just cited, the cover was reinforced by Clinton wire cloth, and no further trouble was experienced.

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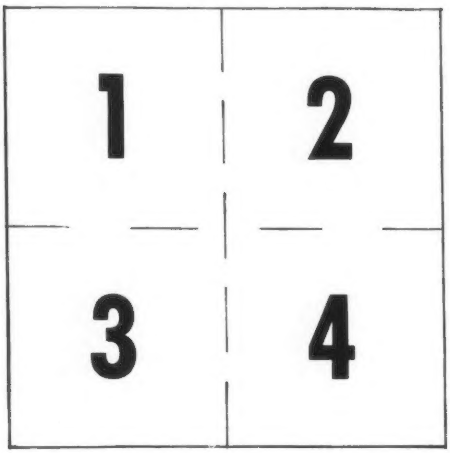

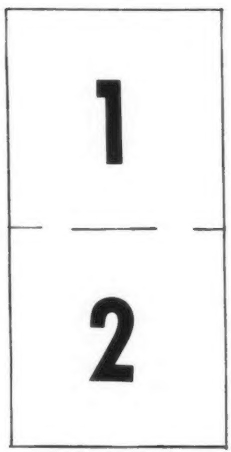

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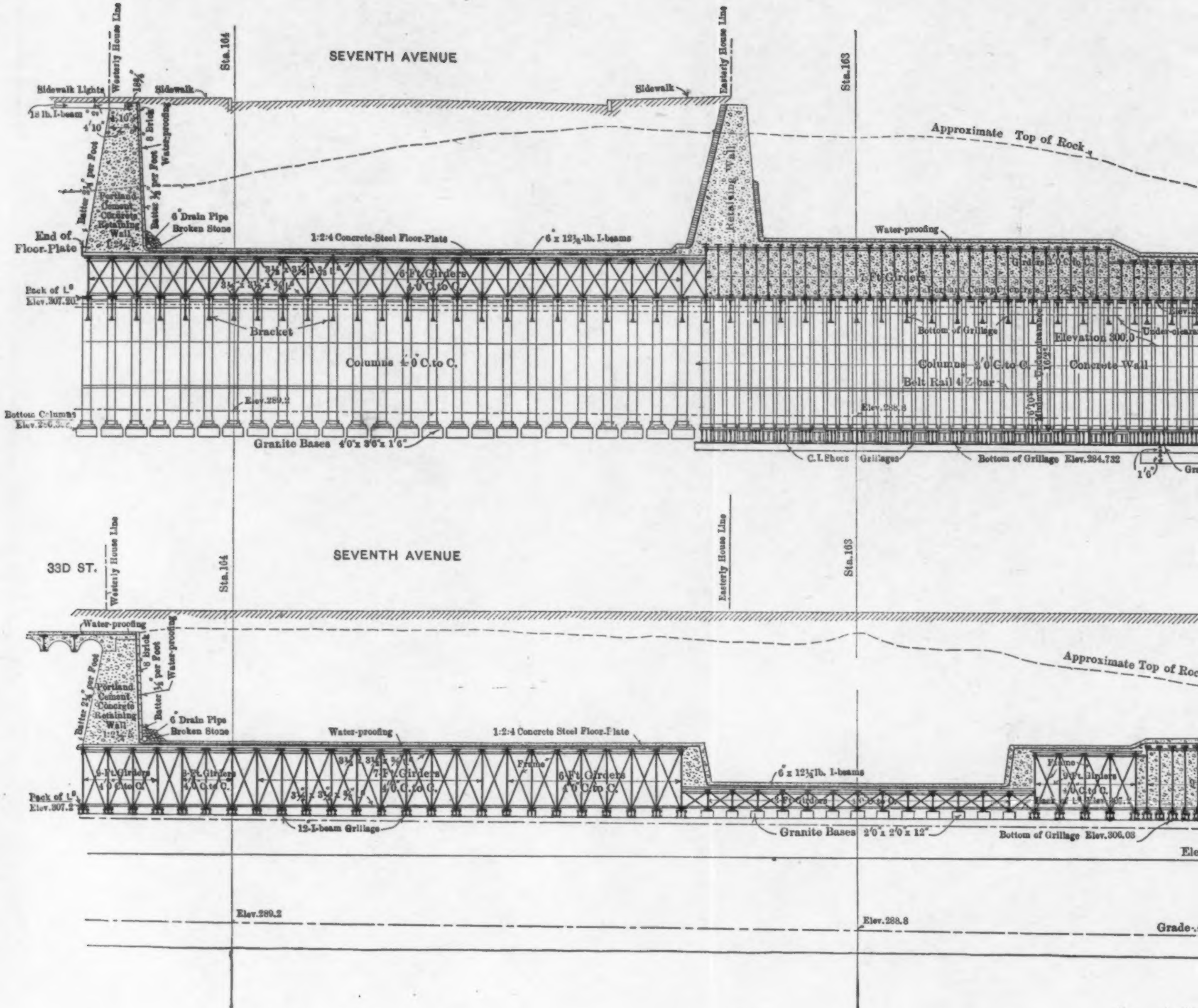
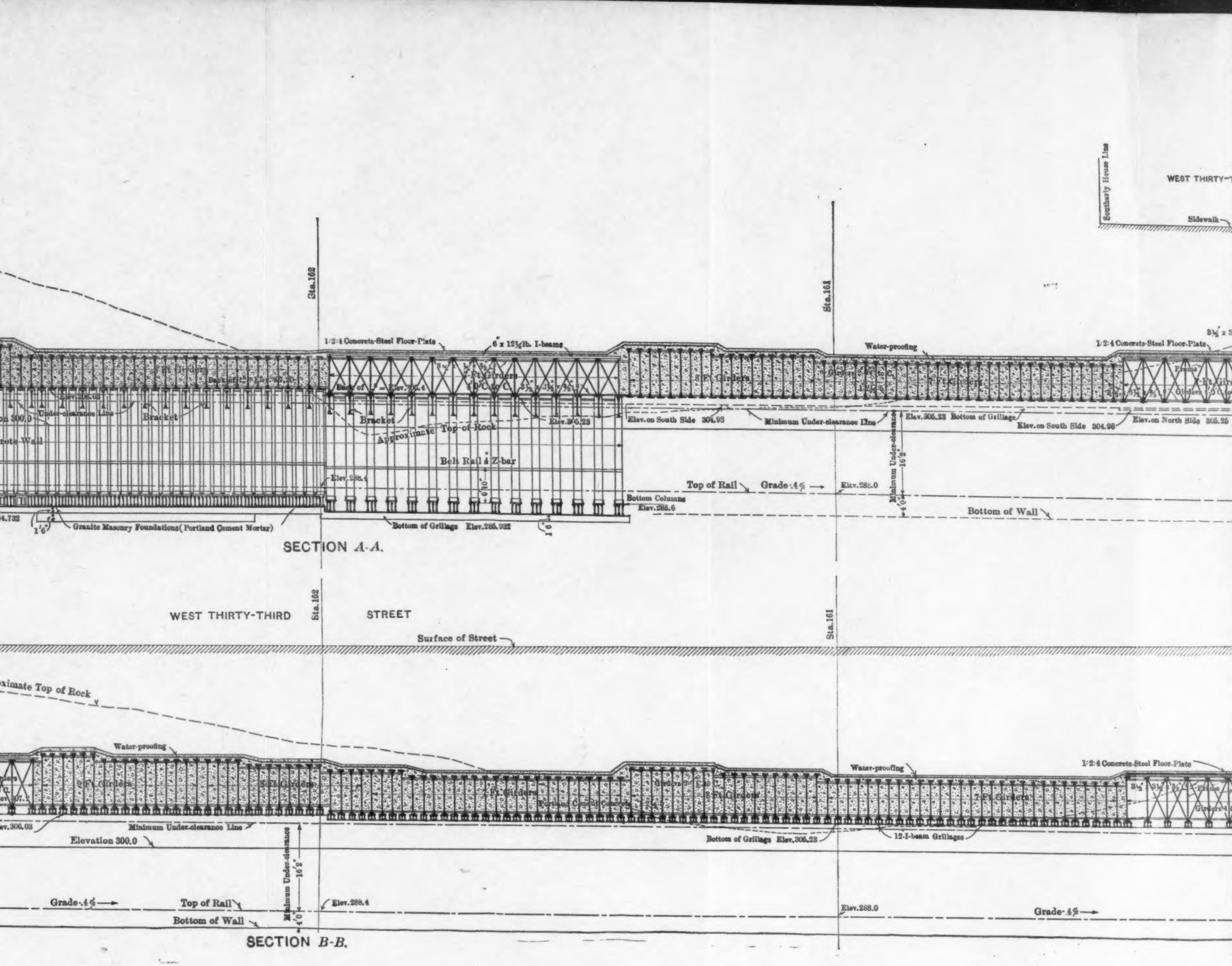
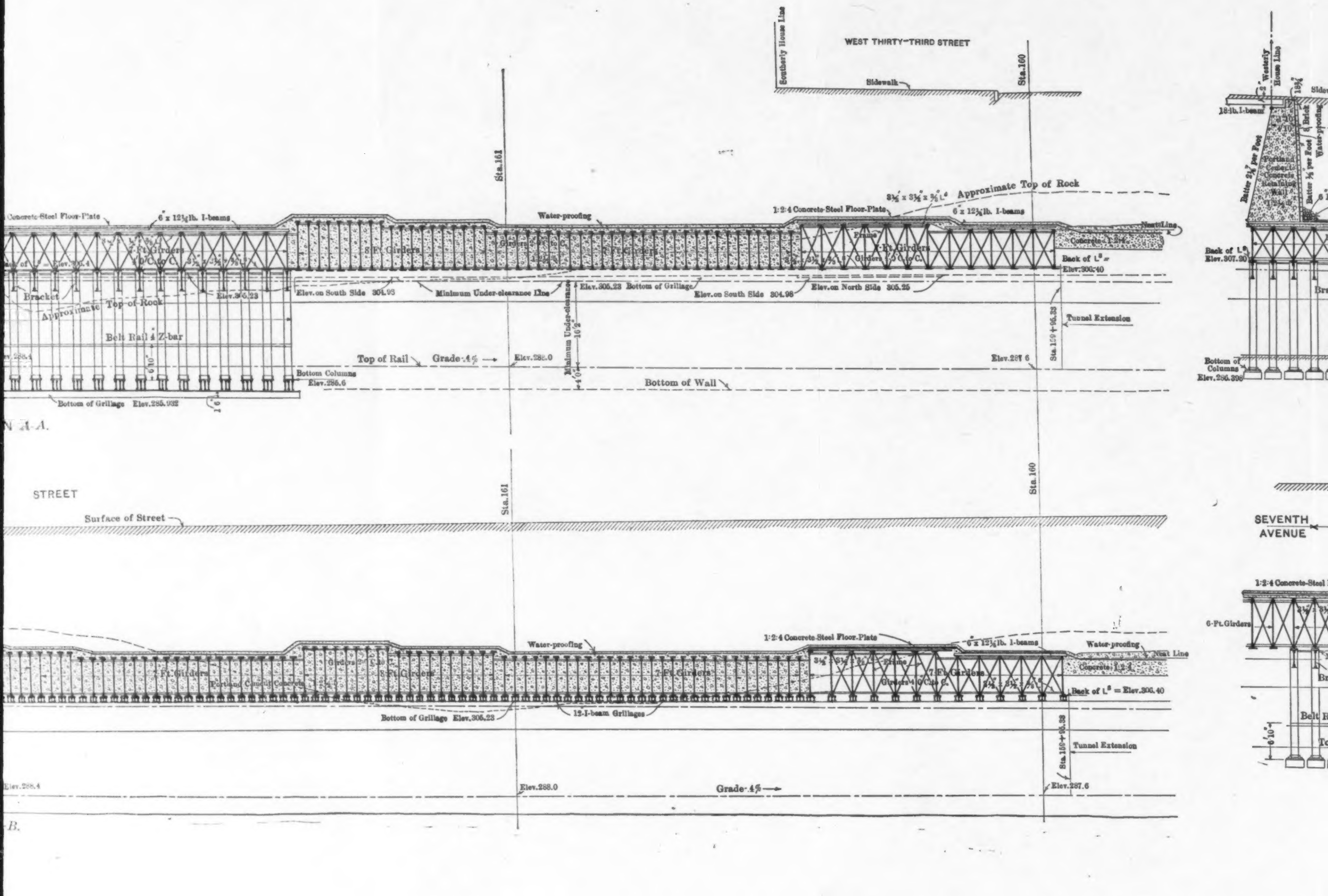


SEVENTH AVENUE
Sta.164
Sta.163
Westerly House Line
Easterly House Line
Sidewalk Lights
Sidewalk
18 lb. I-beam
8" Brick
Water-proofing
Batter 2¼" per Foot
Batter ⅛" per Foot
Portland Cement Concrete Retaining Wall 1:2½:5
6" Drain Pipe
Broken Stone
End of Floor-Plate
1:2:4 Concrete-Steel Floor-Plate
6" x 12¼-lb. I-beams
Approximate Top of Rock
Retaining Wall
Water-proofing
6-Ft. Girders 4'0" C. to C.
Back of L's Elev.307.20
Bracket
Columns 4'0" C.to C.
Bottom of Grillage
Elevation 300.0
Columns 2'0" C. to C.
Concrete Wall
Belt Rail 4" Z bar
Elev.289.2
Elev.288.8
Bottom Columns
Granite Bases 4'0"x 3'6"x 1'6"
C.I.Shoes
Grillages
Bottom of Grillage Elev.284.732
1'6"
33D ST.
SEVENTH AVENUE
Sta.164
Sta.163
Westerly House Line
Easterly House Line
Water-proofing
8" Brick
Batter 2¼" per Foot
Batter ⅛" per Foot
Portland Cement Concrete Retaining Wall 1:2½:5
6" Drain Pipe
Broken Stone
Water-proofing
1:2:4 Concrete Steel Floor-Plate
Approximate Top of Rock
5-Ft. Girders 4'0" C.to C.
7-Ft. Girders 4'0" C. to C.
Frame
6-Ft. Girders 4'0" C. to C.
6" x 12¼lb. I-beams
Back of L's Elev.307.2
12-I-beam Grillage
Granite Bases 2'0" x 2'0" x 12"
Bottom of Grillage Elev.306.08
Elev.289.2
Elev.288.8
Grade

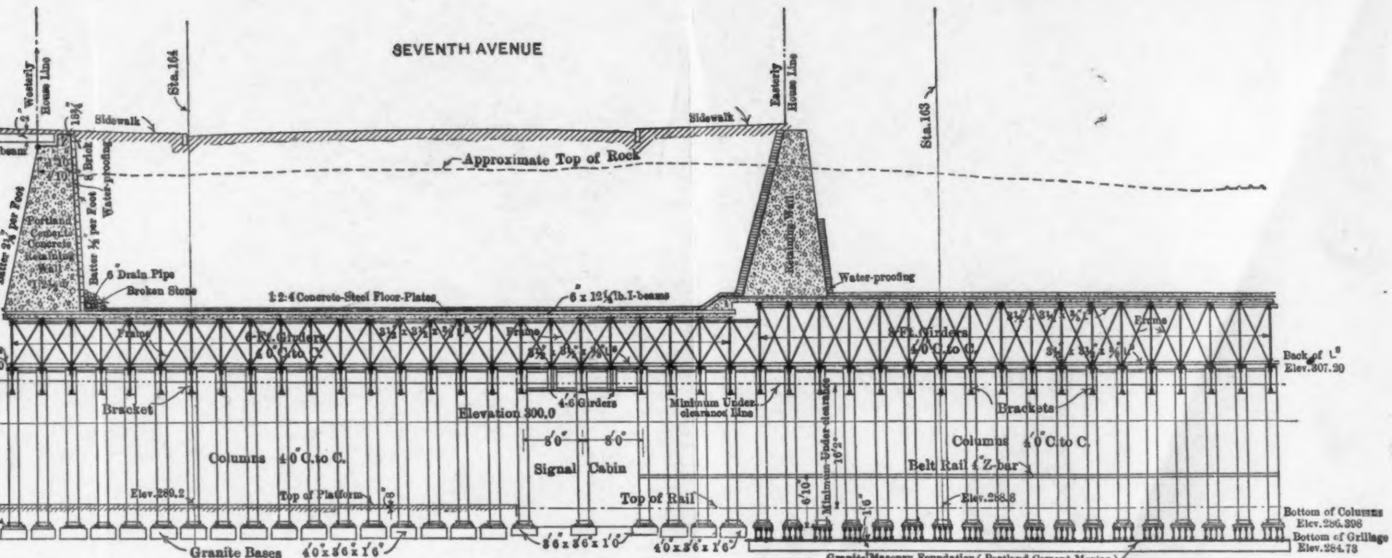


Southerly House Line
WEST THIRTY-T
Sidewalk
Sta.162
Sta.161
1:2:4 Concrete-Steel Floor-Plate
6" x 12¼ lb. I-beams
Water-proofing
1:2:4 Concrete-Steel Floor-Plate
7 Ft Girders
8 Ft Girders
Bracket
Under-clearance Line
Approximate Top-of-Rock
Belt Rail 4" Z-bar
Elev.288.4
Elev.on South Side 304.93
Minimum Under-clearance Line
Elev.305.23 Bottom of Grillage
Elev.on South Side 304.98
Elev.on North Side 305.25
Minimum Under-clearance 16'2"
Top of Rail
Grade .4%
Elev.288.0
Bottom Columns Elev.285.6
Bottom of Wall
Granite Masonry Foundations (Portland Cement Mortar)
Bottom of Grillage Elev.285.932
SECTION A-A.
WEST THIRTY-THIRD
STREET
Surface of Street
Approximate Top of Rock
Water-proofing
8 Ft Girders
7 Ft Girders
Water-proofing
1:2:4 Concrete-Steel Floor-Plate
7 Ft Girders
Minimum Under-clearance Line
Elevation 300.0
Bottom of Grillage Elev.305.23
12-I-beam Grillages
Minimum Under-clearance 16'2"
Grade .4%
Top of Rail
Elev.288.4
Bottom of Wall
Elev.288.0
Grade .4%
SECTION B-B.

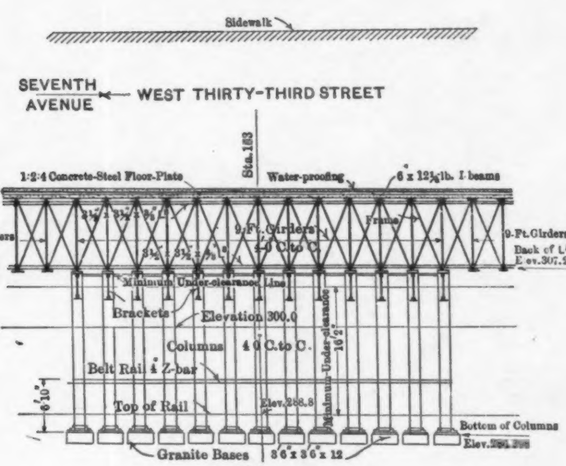


WEST THIRTY-THIRD STREET
Southerly House Line
Sidewalk
Sta.160
Sta.161
Approximate Top of Rock
1:2:4 Concrete-Steel Floor-Plate
6" x 12¼ lb. I-beams
Water-proofing
Concrete-Steel Floor-Plate
8 Ft. Girders
7-Ft. Girders
Back of L⁸ = Elev.306.40
Elev.on South Side 304.98
Minimum Under-clearance Line
Elev.305.23 Bottom of Grillage
Elev.on North Side 305.25
Bracket
Approximate Top of Rock
Belt Rail & Z-bar
Tunnel Extension
Sta.159+95.33
Top of Rail
Grade .4%
Elev.288.0
Elev.287.6
Minimum Under-clearance 16'2"
Bottom Columns Elev.286.6
Bottom of Wall
Bottom of Grillage Elev.285.932
N A-A.
STREET
Surface of Street
Water-proofing
Neat Line
7-Ft. Girders
8-Ft. Girders
Portland Cement Concrete
Concrete 1:2:4
Girders 6' C. to C.
Back of L⁸ = Elev.306.40
Bottom of Grillage Elev.305.23
12-I-beam Grillages
Tunnel Extension
Elev.288.4
Elev.288.0
Grade .4%
Elev.287.6
-B.
Westerly House Line
18-lb. I-beam
Portland Cement Concrete Retaining Wall
Batter 2½" per Foot
Batter ½" per Foot
Back of L⁸ Elev.307.20
Bottom of Columns Elev.286.398
SEVENTH AVENUE
1:2:4 Concrete-Steel
6-Ft. Girders
Belt R

PLATE XLV.
PAPERS, AM. SOC. C. E.
MAY, 1911.
FRANCIS AND O'BRIEN ON
PENNSYLVANIA R. R. TUNNELS: TERMINAL STRUCTURES.



SECTION *C-C*



SECTION *D-D*

EASTERLY PORTION LONGITUDINAL SECTIONS OF THIRTY-THIRD STREET

The columns are encased in continuous concrete fender-walls, with battered faces, from the base of each column to 4 ft. above the top of rail. A reinforcing mesh is placed within 1 in. of the faces of these fender-walls in order to toughen the surface and reduce the tendency to check. Where the columns are at 2-ft. centers, a plumb curtain-wall is built above and monolithic with the battered fender-walls, and the columns are thereby filled and encased with concrete up to the knee-braces. Where the girders are at 2-ft. centers, they are solidly encased in concrete above the bottom flange cover-plates.

A superimposed wall is built along the west side of Seventh Avenue to retain the back-fill. Provision was made for similar walls elsewhere on the street lines, but company ownership of property permitted sloping the back-fill at these points, hence the walls were not built.

All abutment walls and column footings are founded below subgrade on solid rock. All rock bottoms were carefully examined by the engineers and accepted by them before construction was permitted. As the excavation was through solid ledge, the engineers had principally to see that all soft, powder-burned, and loose rock was removed, and jagged, wedge-shaped surfaces cut down before accepting a foundation bottom.

Prior to the execution of the Easterly Portion contract the masonry trunk sewers in Seventh Avenue, 32d and 33d Streets, within the limits of the Easterly Portion, were replaced, at the railroad company's expense, by cast-iron pipe sewers, 30 and 36 in. in diameter. These were maintained in position on falseworks during the conduct of the Easterly Portion contract. Substantial timber bents were left in place beneath them.

Excavation was started immediately after the contract was awarded, in June, 1905, and the procedure was briefly as follows:

Earth excavation was started east of Seventh Avenue, and for the first month was disposed of directly by wagons. An open cut was started west of the car tracks in Seventh Avenue at the end of the first month, and the west roadway was closed to traffic.

Derricks were erected to serve wagons east of Seventh Avenue in the second month. The excavation from the west cut was disposed of by 3-yd. dump-cars hauled to the North River dock by locomotives. The removal of service pipes from the west roadway of Seventh Avenue was started during the second month.

In the third month, a track connection was effected at grade across Seventh Avenue by the use of movable sections at the crossing of the street railway, and wagons were abandoned. A 300-ft. cableway was erected for the excavation of the west roadway of Seventh Avenue. A boiler was set up to serve the drills, and rock excavation was started east of Seventh Avenue.

During the fourth month, the construction of a temporary retaining wall was started along the north side of 33d Street, just north of the north abutment wall. This temporary retaining wall was suggested by the engineers and built by the contractors at their own expense to prevent serious damage to the buildings on the north side of 33d Street. These buildings are founded on the clay and hardpan above the ledge, and, owing to the construction of the temporary retaining wall in front of them, no appreciable settlement was noted.

During the fifth month, a cut-and-cover tunnel was commenced under Seventh Avenue in order to connect the two sections of the work.

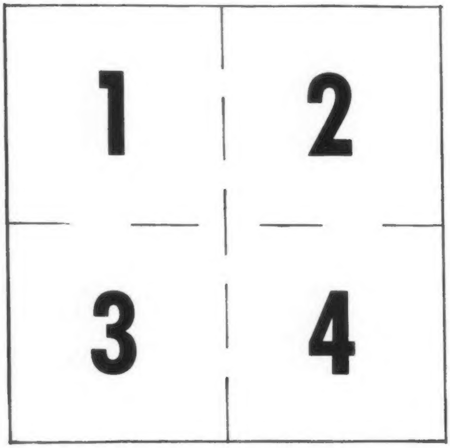
By the middle of the sixth month, the temporary retaining wall above referred to was finished, and the major portion of the north approach east of Seventh Avenue; the west roadway had also been excavated to an average depth of 28 ft. below street level, and a tunnel connection was effected, after which, all excavation was disposed of by way of the contractor's tracks below street level. The use of derricks and skips for serving the dump-cars was continued east of the west track in Seventh Avenue until the completion of the excavation. The cars were served chiefly by steam shovels in the west cut after the sixth month.

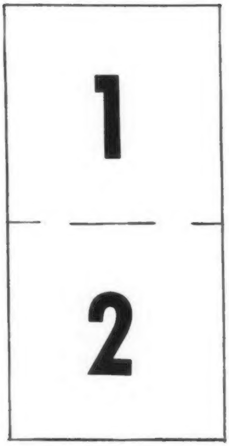
During the seventh month, a battery of boilers was erected south of the north cut east of Seventh Avenue, in order to supply steam for the derricks and drills.

The excavation of the 32d Street approach, east of Seventh Avenue, was started in the eighth month, and the highway was diverted to the company's property on the south.

After the completion of the temporary retaining wall, the north throat cut was widened to include 33d Street; and the highway was thereafter maintained on falseworks. Sub-grade was first reached in one year from the date of beginning excavation, at which time a considerable area in the west roadway of Seventh Avenue and a small area east of that avenue in the 33d Street approach reached bottom.

Maps on this order too large to be en[tirely included in one exposure]
are filmed clockwise beginning in the [upper left hand corner, left to]
right and top to bottom as many frame[s as required. The following]
diagrams illustrate the method:





ntirely included in one exposure
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nes as required. The following

1	2	3
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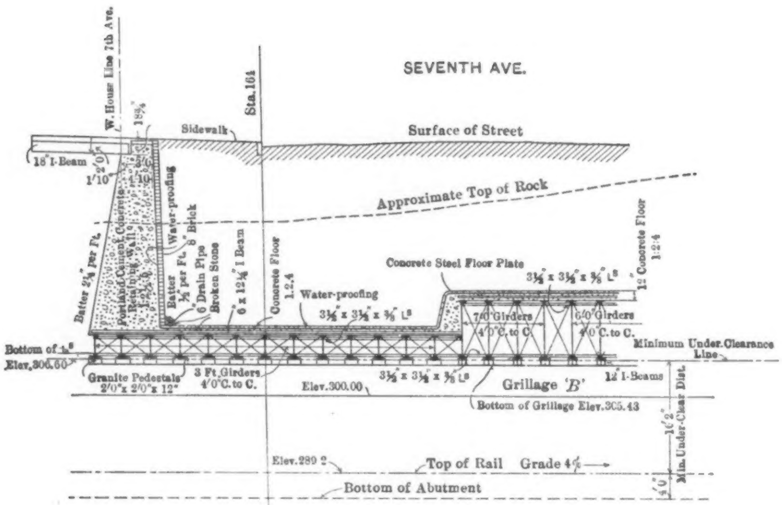
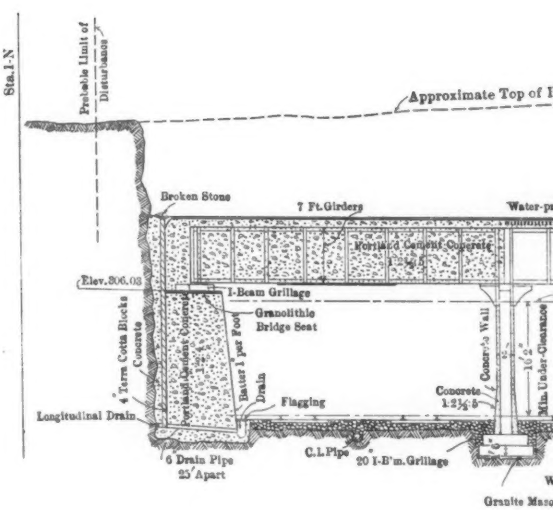
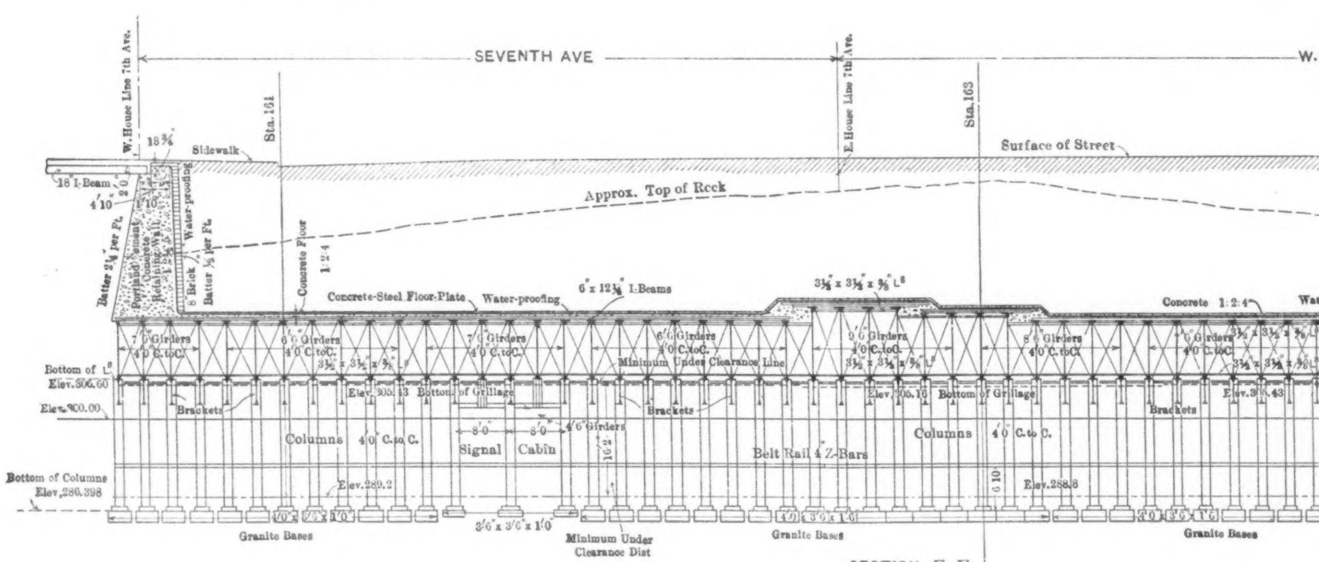
7	8	9

CROSS-SECTIONS ON 32D AND 33D STREETS



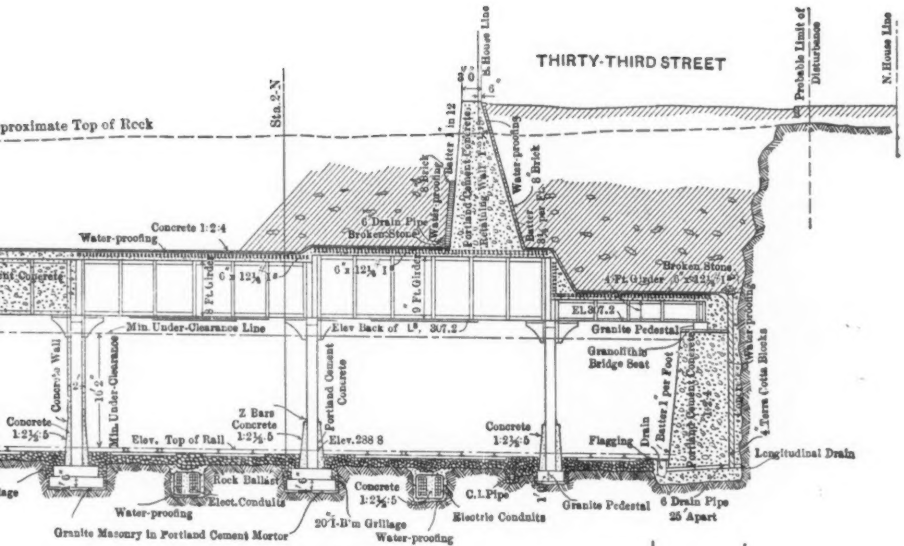
SECTION *E-E*



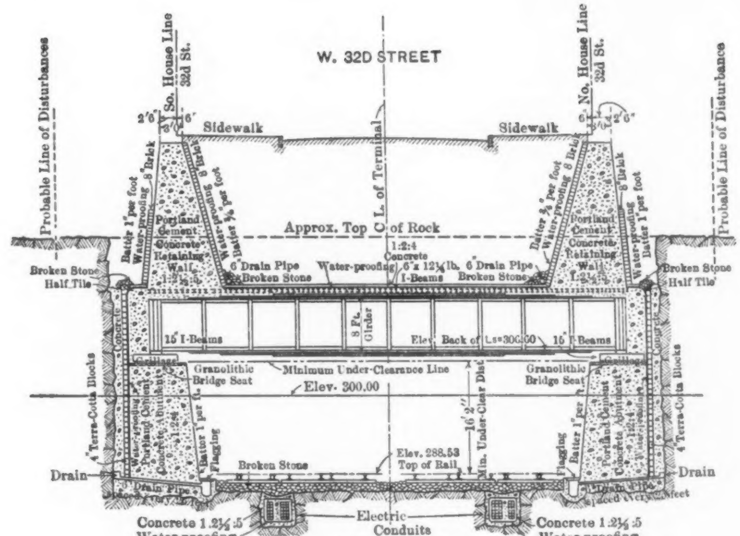


SECTION *F-F*

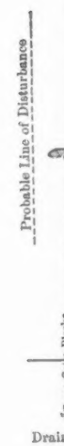
S

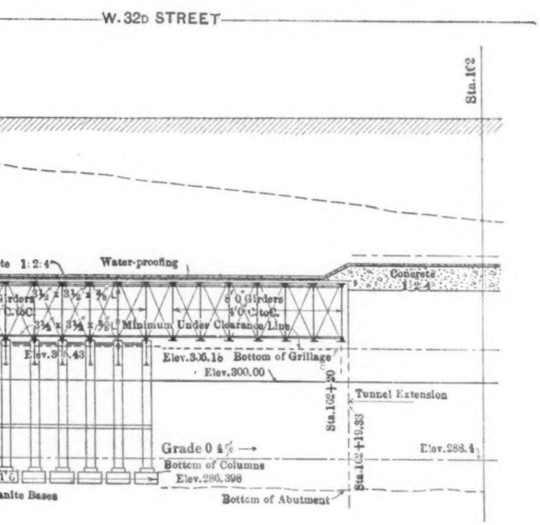


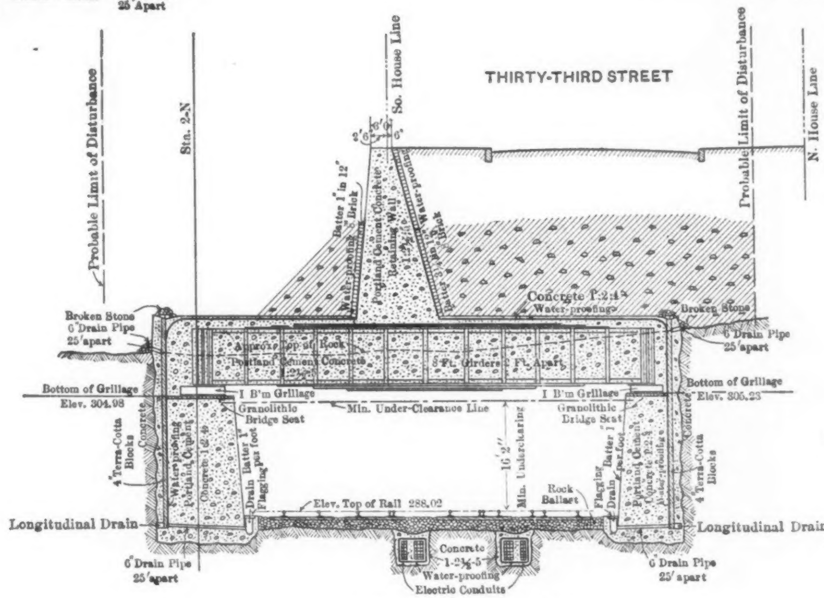
SECTION ON LINE W–W



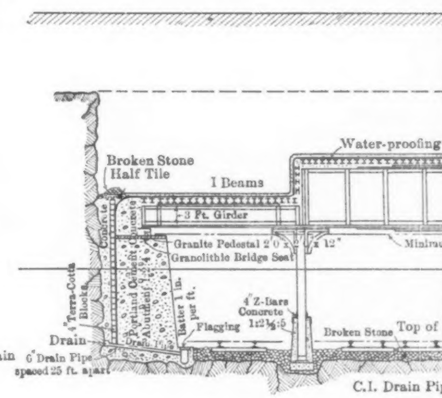
SECTION X-X



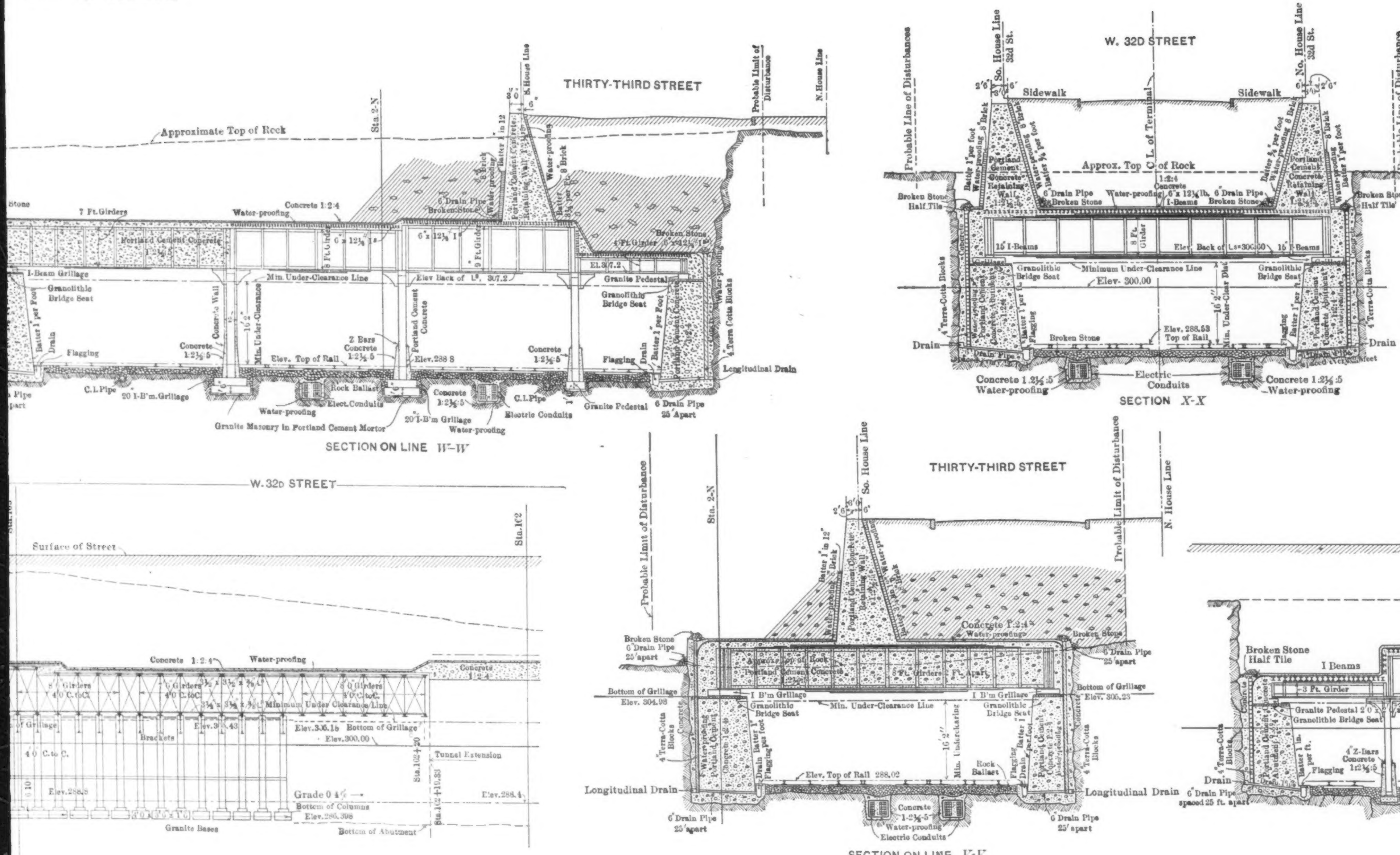


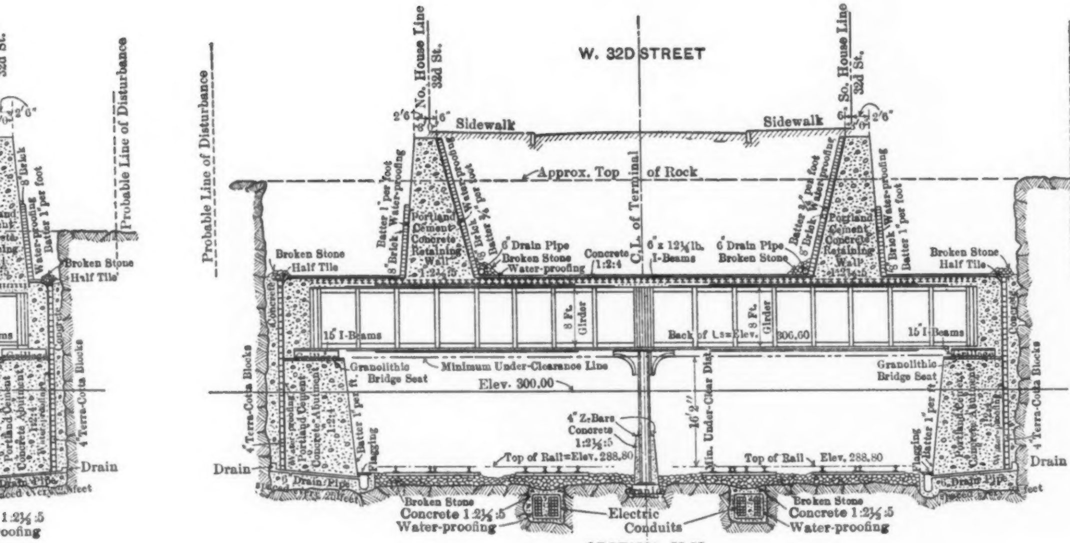


SECTION ON LINE V-V



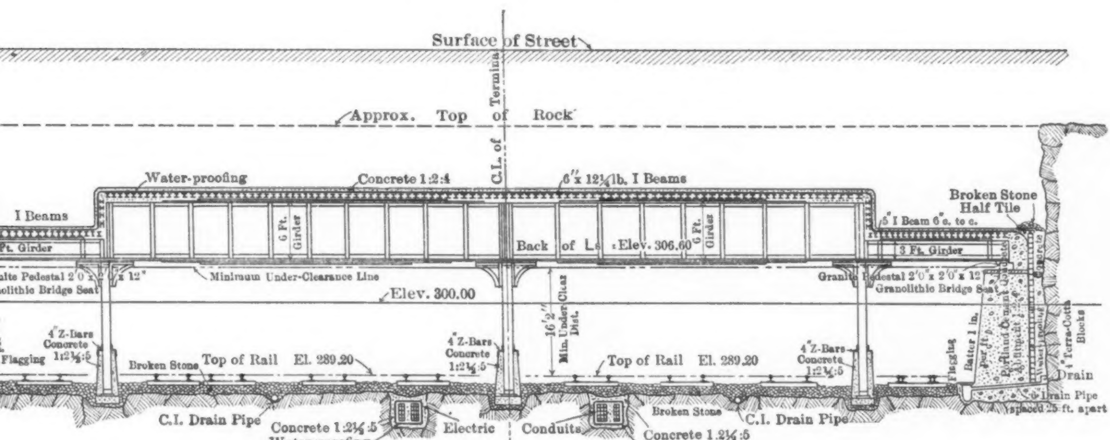
D 33D STREETS





SECTION Y-Y

SEVENTH AVENUE



SECTION Z-Z

Sub-grade was first reached in the 32d Street approach about three months later.

A concrete plant, with two 1-yd. Smith mixers, storage bins, and feed hoppers, was erected east of Seventh Avenue, between the two approaches, in April, 1907. This plant was set above the works, and the materials were delivered to it by a bucket conveyor, the concrete being discharged into 1-yd., metal dump-cars, by which the concrete was distributed to the various works on industrial tracks placed on the forms and on a trestle built for the purpose. A section of backing wall for the south abutment, the 33d Street approach, and the foundations of the intermediate bents for this approach were started east of Seventh Avenue in May, 1907, and the first section of abutment wall was completed in that month.

The abutment walls for the south side of the 33d Street approach and the north side of the 32d Street approach west of the car tracks in Seventh Avenue, also the foundations for the center bent of the 32d Street approach west of the Seventh Avenue car tracks, were built during June, 1907. All abutments and foundations were partly completed during July, 1907, and the steel erection was started at the south end of the 32d Street approach west of the car tracks in Seventh Avenue. The steel erection for the 32d Street approach west of the car tracks in Seventh Avenue was completed on September 1st, 1907. The steel erection for the 33d Street approach east of Seventh Avenue was started on September 1st, 1907. The permanent structure was wholly completed west of the Seventh Avenue car tracks in December, 1908.

A temporary timber trestle was erected on the deck of the completed portion of the permanent structure, and the street-railway and other traffic in Seventh Avenue was diverted to this trestle on February 13th, 1908, after which the rock core which had been left beneath the street-railway tracks in the middle of Seventh Avenue was excavated. The excavation was entirely completed on September 30th, 1908. The abutment walls and foundations were completed on August 1st, 1908. The steel erection was completed about September 30th, 1908. All the floor-plate work, water-proofing, and protecting masonry were completed on October 6th, 1908. The back-filling for the restoration of the highways was started on the 33d Street approach east of Seventh Avenue on April 7th, 1908. A trestle was built over the permanent

bridging in Seventh Avenue, at the railroad company's expense, for the support of the street-railway tracks, the sewers, and the water and gas mains, in their permanent positions, and, after back-filling had been placed to the top of this trestle, the street railway was restored to its original location on January 5th, 1909, and four days later the east roadway of Seventh Avenue was paved and restored to traffic, thus permitting the completion of the west roadway, which was finished in May, 1909. The pavement on 33d Street east of Seventh Avenue was completed in January, 1909, and that on 32d Street east of Seventh Avenue in December, 1908. The company's property was fenced in, thus marking the virtual completion of this contract, about June 15th, 1909, four years from the date of beginning.

Owing to the heavy fill for the restoration of the streets and avenue it was determined to delay placing the final pavement until at least one year after the highways had been first restored to traffic.

Extracts from the specifications for the Easterly Portion are given in Appendix A. Extracts from the specifications for the manufacture of the steel for the Easterly Portion are given in Appendix B.

VIADUCTS.

The Eighth Avenue Viaduct.—This viaduct is 525 ft. long, 100 ft. wide, and extends entirely across the terminal between 31st and 33d Streets. Top of rail is about 39 ft. below street level at the south end of the viaduct and about 41 ft. below street level at the north end. After allowing for a minimum under-clearance of 16 ft. 2 in. and a minimum depth of 19 ft. from street level to top of structure, as required by the franchise, the permissible depth of structure over tracks was found to vary from 3 ft. 6 in. at the south end to 5 ft. 6 in. at the north end.

For the reasons stated under the heading "Easterly Portion," it was determined to restore the highway by back-filling over the viaduct roof, hence the latter was designed to support the weight of back-fill plus a uniform live load of 300 lb. per sq. ft., resulting in a total load of about 2 700 lb. per sq. ft.

Owing to the shallow working depths, the bents were located in all platforms and between tracks wherever possible. The resulting spans vary from 20 ft. 5 in. to 43 ft. 9 in. from center to center of bearings.

A riveted, face-connected, plate-girder structure, with riveted,

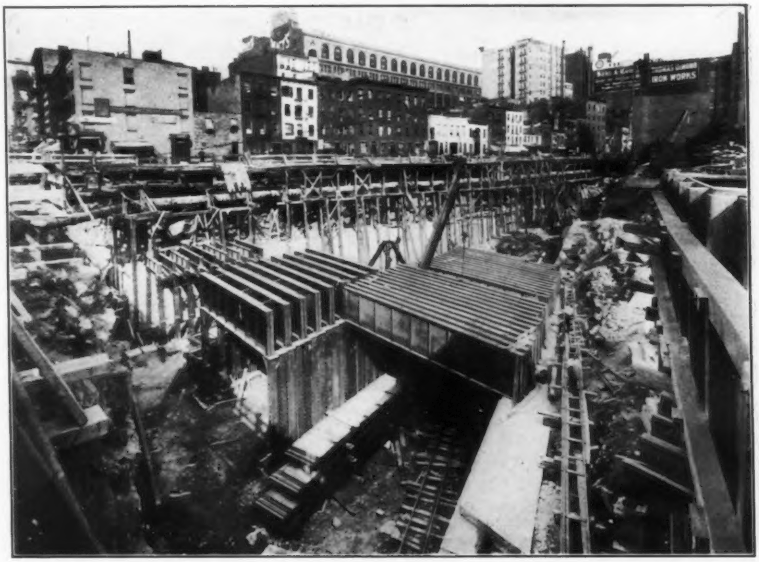


FIG. 1.—STEEL ROOFING, EASTERLY PORTION.

FIG. 2.—STEEL ROOFING AT THROAT, EASTERLY PORTION.

I-shaped columns, was adopted. The floor-plate is of 4-in. I-beams laid transversely, spaced from 6 to 8 in. on centers, spanning at least three girders, and breaking joints every sixth beam; it is similar in type to that described under the heading "Easterly Portion."

Superimposed retaining walls extend from the floor-plate to the street level on both faces of the viaduct; the basin thus formed is water-proofed, covered with masonry, and contains the back-filling.

The columns are uniformly spaced 10 ft. from center to center transversely of the viaduct; each is built up of one web-plate, 16 in. wide, four 8 by 6-in. L's, and from two to six cover-plates, 18 in. wide, having open holes at the top for girder connections, and riveted plate and fitted stiffener angle bases. The columns bear on cut granite cap-stones underpinned to solid rock, except as hereinafter noted, by concrete piers. Each column base is secured to the granite cap-stones by two 1¼-in. fox-bolts, 12 in. long.

Face-connected girders, varying in depth from 5 ft. to 7 ft. 2½ in., each built up of one web-plate, with four side-plates where required, and four 6 by 6-in. L's, and provided with open holes through riveted fillers for web connections with longitudinal girders, are framed between the columns transversely, thus completing the bents.

The longitudinal girders vary in depth from 2 ft. 4½ in. to 4 ft. 4½ in., and are 3 ft. 4 in. from center to center in the outer bays under the retaining walls, and 5 ft. from center to center between the outer bays. They are framed between the columns and transverse girders of the bents above described. Each longitudinal girder is built up of one web-plate, with four side-plates where required, four 6 by 6-in. or four 8 by 6-in. flange L's, and from two to six cover-plates.

The site of the viaduct was excavated to sub-grade, a timber trestle was erected to maintain the highway for traffic, and the north and south abutments were built under another contract.

Some interesting incidents of the construction of this viaduct are the following: Top of rock was found below track level over the greater portion of the south half of the viaduct, gradually rising to the surface of the avenue in the north half. Pockets of disintegrated feldspar, having the consistency of thoroughly compacted ashes, and saponifying in water, were encountered in several instances immediately beneath the viaduct foundations of the six southerly bents. Wash-borings at eighteen column locations showed the depth to solid

rock in these pockets to vary from 5 ft. to more than 30 ft. below sub-grade. After borings had been made, excavation was carried deep enough in all soft material to disclose the character and extent of the pockets, and further explorations were made, where required, with rock drills. In several instances the pockets were found to extend over the entire area of the column foundation, the depth to firm rock being more than 25 ft. below sub-grade. In such cases spread footings were built, based on conservative bearing values determined according to the conditions encountered in each case. In other instances the pockets tapered so that their area at moderate depths was found to be only a small percentage of the area of the piers. In these cases excavation was made until firm rock was encountered at depths of 6 to 16 ft. below sub-grade. In many other cases the rock bottom was badly seamed with mica schist and disintegrated rock, and in such instances spread footings were provided, based on the usual bearing values for hardpan.

The timber trestle occupied the middle of the viaduct site for a width of about 40 ft., hence almost half of the steel had to be placed through this trestle, and the latter had to be constantly readjusted, first during the building of foundations, then during the placing of steel, and afterward during the restoration of the service mains, etc.

Owing to the type of the permanent structure, it was hoped that the steelwork could be erected by working simultaneously in both directions from the center bent of the viaduct, but it happened that the south half was ready for the steelwork first, and therefore the erection was started at that end and continued steadily northward. The easterly bays were set in advance, and the remaining bays were filled in just back of them until the center bent was reached. The structure was constantly measured and plumbed, and riveted, but, when all the steel had been placed south of the center bent, check measurements were made and complete data obtained as to the position of the structure. It was found to be plumb and varying from $\frac{1}{4}$ to $\frac{7}{16}$ in. south of its true position at the center. Fillers were accordingly inserted as required on the north face of the center bent, and the erection proceeded, closing on the north abutment within $\frac{1}{4}$ in. of true position. The steel was erected in temperatures varying from about 20 to 90° Fahr., and the only provision against creep was the customary shortening of the girders from $\frac{1}{16}$ to $\frac{1}{8}$ in.

STEEL VIADUC

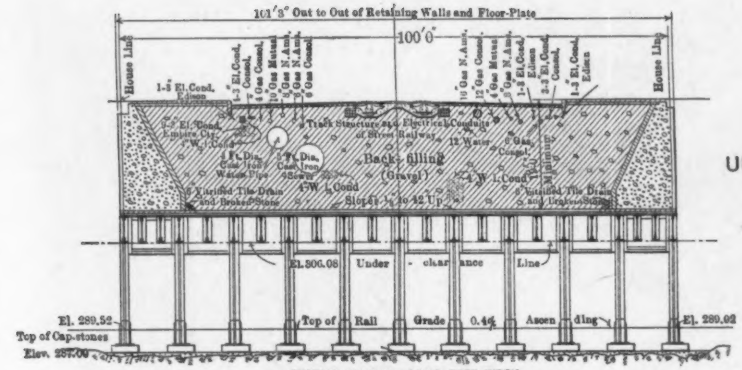
UNDER EIGHTH AV

PLAN AND SECTIO

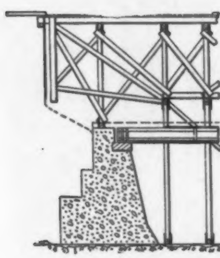
SHOWING RELATION

TEMPORARY TO PERMA

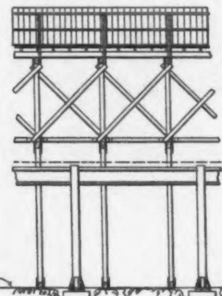
STRUCTURE



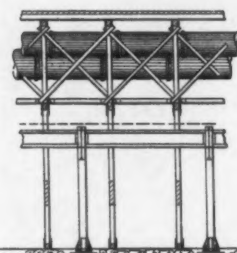
SECTION SHOWING COMPLETED WORK



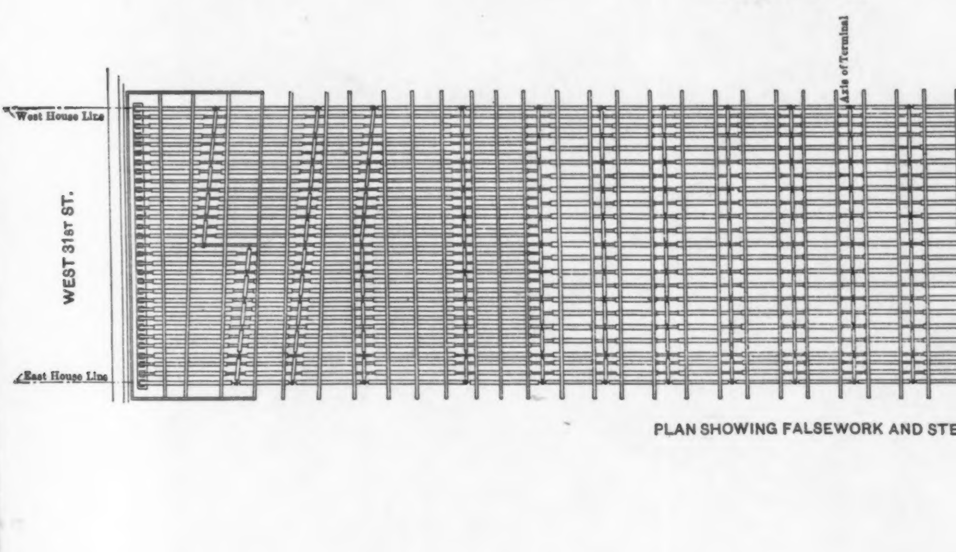
ELEVATION AT SOUTH END.



TYPICAL ELEVATION.



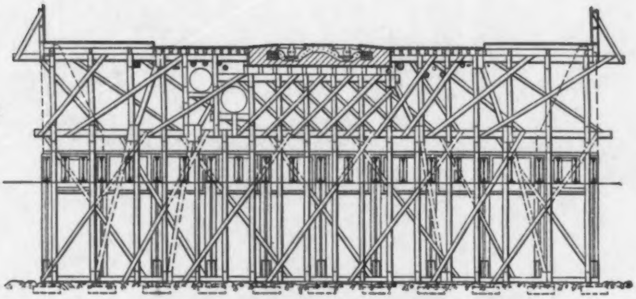
LONGITUDINAL SECTION WEST OF SEWER AND WATER PIPES.



PLAN SHOWING FALSEWORK AND STE

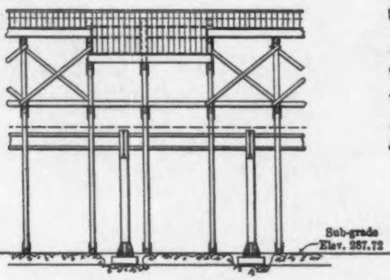
PLATE XLVIII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
FRANCIS AND O'BRIEN ON
PENNSYLVANIA R. R. TUNNELS: TERMINAL STRUCTURES.

TEEL VIADUCT
EIGHTH AVEUNE
LAN AND SECTIONS
OWING RELATION OF
ORARY TO PERMANENT
STRUCTURE

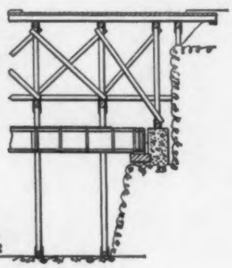


SECTION SHOWING FALSEWORK AND STEELWORK IN POSITION.
Note:- Timber Members that Interfere with Erected Position of Steel Girders are shown Dotted.

WEST OF
PIPES,



LONGITUDINAL SECTION AT STREET RY.
MANHOLE JUST EAST OF CTR. LINE.



ELEVATION AT NORTH END.

EWORK AND STEELWORK IN POSITION.

The back-filling, for the restoration of the highway, was advanced from south to north, in bench formation, as rapidly as the masonry cover was completed over the water-proofing. About at the center of the viaduct the back-filling was advanced too close upon the green masonry, and a settling down and bulging of the 8-in. backing wall was noted, accompanied by vertical cracks, due apparently to the slipping of the back-filling placed against the wall. To overcome the difficulty, the thickness of the backing was changed from 8 to 4 in., and a substantial footing was provided. No recurrence of the trouble was experienced after the section of the brick backing was modified. Portions of the timber trestle were left in place for the support of the street-railway tracks and the service mains. The street railway was jacked up, lined, and wedged to grade on this trestle support prior to paving. The highway was restored at first by granite block pavement in sand, and this temporary pavement was maintained in serviceable condition during the settlement of the back-fill. Permanent asphalt pavement will be provided eventually.

The sidewalks are of granolithic, top-dressed concrete on a cinder base, with cut stone curbs.

The excavation for the foundations was started in November, 1905, and completed in June, 1907. The foundation masonry was started in December, 1905, and completed in July, 1907. The steel erection was started in September, 1906, and completed in August, 1907. The superstructure masonry was started in May and completed in November, 1907. The back-filling for the restoration of the highway was started in July and completed in December, 1907. The temporary pavement was completed and the highway entirely restored to traffic in March, 1908, $2\frac{1}{2}$ years after beginning work.

The 31st and 33d Street Viaducts.—These structures support the highways along the north and south margins of the terminal site, between Seventh and Eighth Avenues, between Eighth and Ninth Avenues, and along 31st Street for a distance of 200 ft. west of Ninth Avenue.

The franchise permitted the tops of these structures to be placed 2 ft. 6 in. below the street surface, except south of the south curb line of 31st Street and north of the north curb line of 33d Street, where it required a minimum depth of 5 ft.

The 31st Street Viaduct, between Seventh and Eighth Avenues, is 800 ft. long, 81 ft. wide for a distance of 160 ft. in front of the

Service Building, and 58 ft. wide elsewhere. It is supported on the north by the Station Building steelwork and on the south by a retaining wall, except in front of the Service Building, where it is supported by the steelwork of the latter. This viaduct has two main decks; the upper deck, in front of the Service Building, supports the highway and both sidewalks; elsewhere the upper deck supports a portion of the highway and the north sidewalk. Beneath the roadway deck there is a baggage passageway connecting the Seventh and Eighth Avenue ends of the Station Building. This baggage passageway deck also includes a carriage return driveway, just south of the general waiting-room, and a portion of the main concourse floor for the Station Building. It also includes a floor immediately in front of the Service Building for the support of apparatus.

Under a portion of the baggage passageway floor a pipe gallery has been constructed for service lines from the Service Building to the Station Building.

A column line was established just south of the north house line of 31st Street, and there are intermediate column supports for the viaduct opposite the columns of the south wall of the Station Building, and spaced at varying intervals from 11 ft. 8 in. to 30 ft. $6\frac{1}{2}$ in. from center to center. Columns were also located north of the Service Building for the support of the southerly margin of the highway and the baggage passageway.

The principal columns are made up of one $16\frac{1}{2}$ by $\frac{7}{16}$-in. web-plate, four 6 by $3\frac{1}{2}$ by $\frac{3}{4}$-in. angles, and two 15-in. 33-lb. channels, reinforced below the baggage passageway floor level, by two 12 by $\frac{3}{8}$-in. cover-plates, provided with open holes for the connection of the girders and beams of the baggage passageway floor and for girder connections at the top, and provided with riveted plate and stiffener angle bases. Riveted plate and angle brackets are provided at all connections between transverse girders and columns.

All but three of the columns bear on cut granite cap-stones underpinned to solid rock by concrete piers. Three of the columns rest on steel I-beam grillages underpinned to solid rock by concrete piers. The latter three column foundations are located between the conduit lines, where the available space did not permit of the economical use of stone templates. The foundation concrete is composed of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts broken stone.

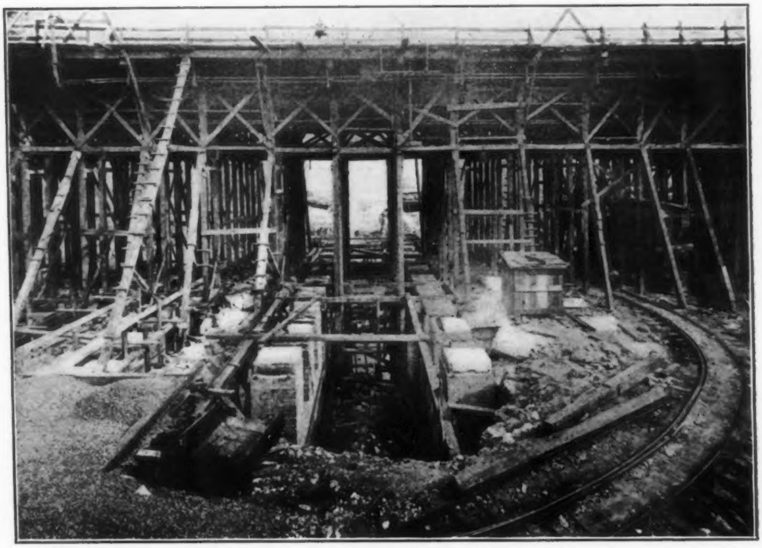


FIG. 1.—TEMPORARY BRIDGE, PERMANENT FOUNDATIONS, AND TRUCKING SUBWAY, EIGHTH AVENUE.

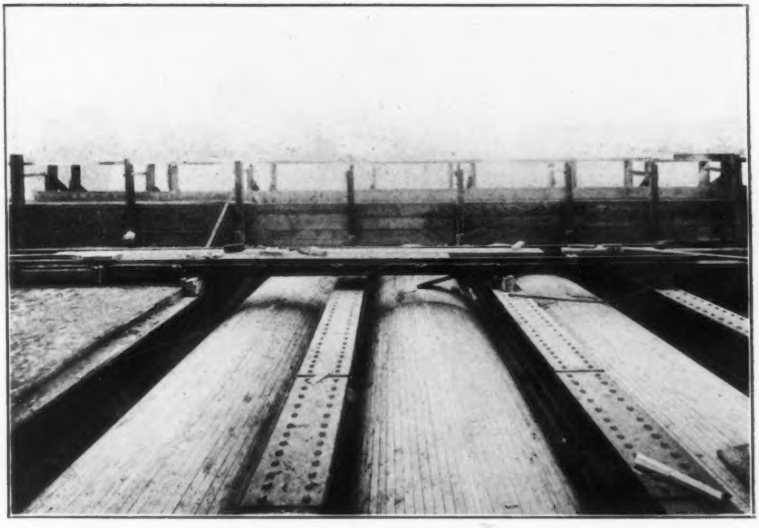


FIG. 2.—THIRTY-FIRST STREET BRIDGE FLOOR.

The deck immediately beneath the roadway is of plate-girder construction, the sidewalks being supported on I-beam framing. The sidewalk beams and roadway girders are approximately at 5-ft. centers, and set transversely of the viaduct. The roadway girders are face-connected on the north to longitudinal girders which frame between the viaduct columns, and are similarly connected on the south in front of the Service Building, but elsewhere on the south they are supported on the retaining wall bridge seat. These roadway deck girders are made up of one 24-in. web-plate, four 6 by 6-in. angles, and two 14-in. cover-plates. Between the roadway girders three-center spandrel arches were turned, with haunches bearing on the bottom flanges of the girders. These arches are of 1:2:4 Portland cement concrete, and are 12 in. thick at the crown with tops leveled off 3 in. above the tops of the girders. This top surface is water-proofed with six-ply pitch and felt, connected to the water-proofing of the retaining wall, and turned up elsewhere back of the curbs. A protecting cover of 1:2½:5 Portland cement concrete, 4 in. thick, is placed over the water-proofing. The back-filling was placed over this cover, and the roadway was temporarily paved with granite blocks in sand. A temporary timber curb and bulkhead was constructed just south of the north curb line to retain the fill and pavement pending the setting of special curb and sidewalk lights in front of the Station Building. This temporary curb was subsequently used as the sill of the temporary fence enclosing the station site.

The baggage passageway floor is of steel-beam, plate-girder, and lattice-girder construction. The floor-beams vary from 15-in., 45-lb. I's to 24-in., 80-lb. I's; they are spaced about 5 ft. from center to center, and are set longitudinally, resting on the top chords of the transverse lattice girders of the pipe gallery; they are face-connected to the transverse plate girders elsewhere, with the tops of the beams set 1 in. below the girders.

The pipe gallery floor-beams vary from 10-in., 25-lb. I's to 12-in., 31½-lb. I's; they are spaced about 5 ft. from center to center, and are set longitudinally and face-connected to the transverse girders with the bottom of the beams 1 in. above the bottom flanges of the girders. Between and over the floor-beams of the pipe gallery and baggage passageway there are reinforced concrete floors, the slabs varying in thickness from 4 to 6 in., with beveled haunches footed on beam

flanges. These floors are of 1:2:4 Portland cement concrete (using $\frac{3}{4}$-in. stone), reinforced by Clinton wire cloth. The bottoms of the slabs are 1 in. below the tops of the beams. These reinforced concrete floors have a granolithic top dressing. The north side of the baggage passageway and pipe gallery and the southerly margin in front of the Service Building are enclosed by brick walls supported on the viaduct steelwork. A granolithic sidewalk, 2 ft. 6 in. wide, with Wainwright, steel-bow, concrete curb, 7 in. high, is laid on the baggage passageway floor against the north wall, and there is a quarter-round granolithic fender curb of 12-in. radius on the south side of the baggage passageway floor. The general excavation to sub-grade, the construction of the retaining wall, the Service Building, and the Station Building were done under other contracts.

The excavation for the forty-three column foundations of this viaduct was started in June and completed in November, 1906. The foundations were started in April and completed in December, 1906. The steel erection was started in November, 1906, and completed in March, 1907. The superstructure masonry was started in March and completed in July, 1907. The temporary pavement was completed and the highway restored to traffic in November, 1907.

The 33d Street Viaduct, between Seventh and Eighth Avenues, is 800 ft. long, 78 ft. wide for a distance of 297 ft. opposite the general waiting-room, and 57 ft. wide elsewhere.

It is similar in type to the 31st Street Viaduct between Seventh and Eighth Avenues, hereinbefore described, but the lower deck in this case is a part of the exit concourse, and is arranged for future connections to rapid transit subways in Seventh and Eighth Avenues and 34th Street. The governing floor grades and the under-clearance left a permissible depth of only about 2 ft. for the lower deck. It was determined, therefore, to frame this lower deck transversely between the intermediate column line and the north retaining wall, and, opposite the general waiting-room, to the north column line. The floor-beams for the shorter spans vary from 15-in., 42-lb. I's to 18-in., 60-lb. I's. The long-span floor-beams are built up of 18-in. web-plates and four 6 by 4-in. L's, and 24-in. web-plates with four 6 by 6-in. L's. The floor-beams are spaced from 3 ft. 6 in. to 5 ft. from center to center, are face-connected to the longitudinal girders and columns on the column lines, and set into pockets in the ledge beneath the north retaining wall

flanges. These floors are of 1:2:4 Portland cement concrete (using $\frac{3}{4}$-in. stone), reinforced by Clinton wire cloth. The bottoms of the slabs are 1 in. below the tops of the beams. These reinforced concrete floors have a granolithic top dressing. The north side of the baggage passageway and pipe gallery and the southerly margin in front of the Service Building are enclosed by brick walls supported on the viaduct steelwork. A granolithic sidewalk, 2 ft. 6 in. wide, with Wainwright, steel-bow, concrete curb, 7 in. high, is laid on the baggage passageway floor against the north wall, and there is a quarter-round granolithic fender curb of 12-in. radius on the south side of the baggage passageway floor. The general excavation to sub-grade, the construction of the retaining wall, the Service Building, and the Station Building were done under other contracts.

The excavation for the forty-three column foundations of this viaduct was started in June and completed in November, 1906. The foundations were started in April and completed in December, 1906. The steel erection was started in November, 1906, and completed in March, 1907. The superstructure masonry was started in March and completed in July, 1907. The temporary pavement was completed and the highway restored to traffic in November, 1907.

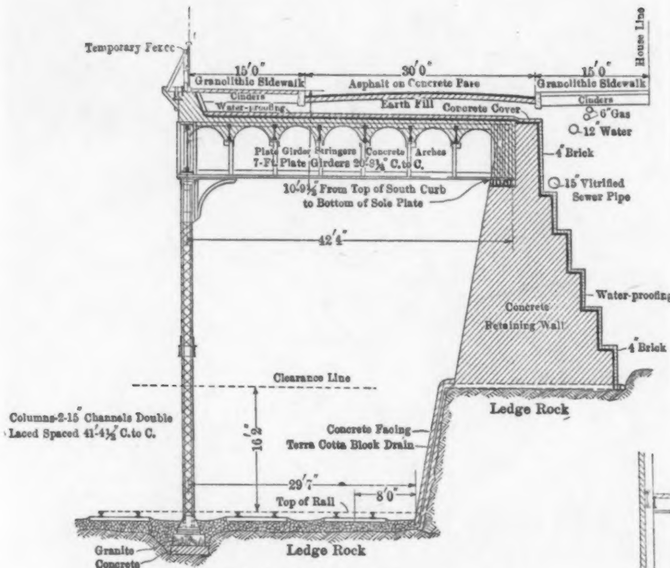
The 33d Street Viaduct, between Seventh and Eighth Avenues, is 800 ft. long, 78 ft. wide for a distance of 297 ft. opposite the general waiting-room, and 57 ft. wide elsewhere.

It is similar in type to the 31st Street Viaduct between Seventh and Eighth Avenues, hereinbefore described, but the lower deck in this case is a part of the exit concourse, and is arranged for future connections to rapid transit subways in Seventh and Eighth Avenues and 34th Street. The governing floor grades and the under-clearance left a permissible depth of only about 2 ft. for the lower deck. It was determined, therefore, to frame this lower deck transversely between the intermediate column line and the north retaining wall, and, opposite the general waiting-room, to the north column line. The floor-beams for the shorter spans vary from 15-in., 42-lb. I's to 18-in., 60-lb. I's. The long-span floor-beams are built up of 18-in. web-plates and four 6 by 4-in. L's, and 24-in. web-plates with four 6 by 6-in. L's. The floor-beams are spaced from 3 ft. 6 in. to 5 ft. from center to center, are face-connected to the longitudinal girders and columns on the column lines, and set into pockets in the ledge beneath the north retaining wall

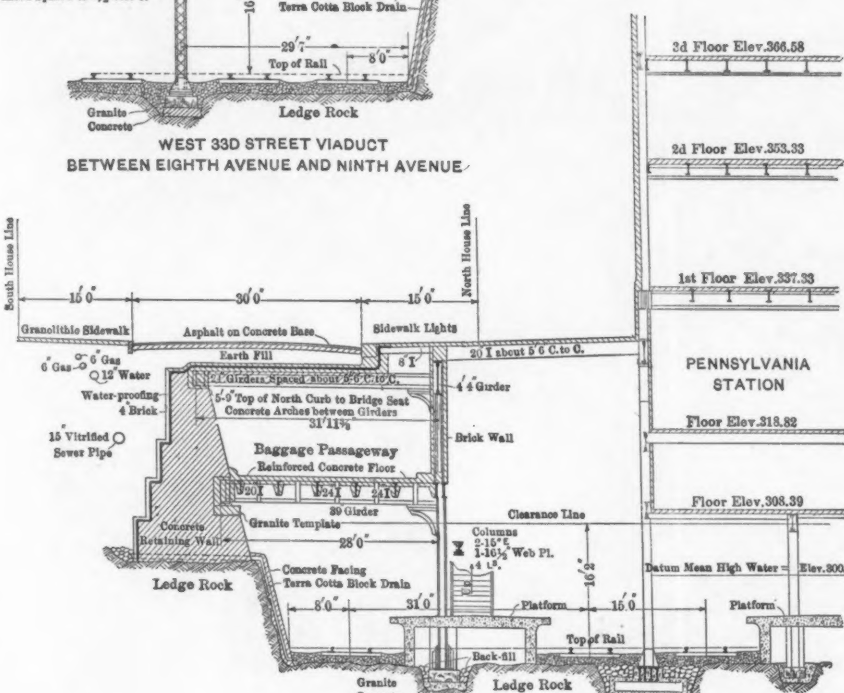
TYPICAL SECTIO

OF VIADUCTS

WEST 31ST AND WEST 3



WEST 33D STREET VIADUCT
BETWEEN EIGHTH AVENUE AND NINTH AVENUE



WEST 31ST STREET VIADUCT
BETWEEN SEVENTH AVENUE AND EIGHTH AVENUE

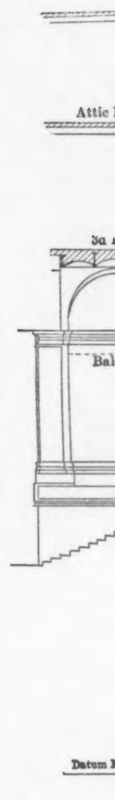
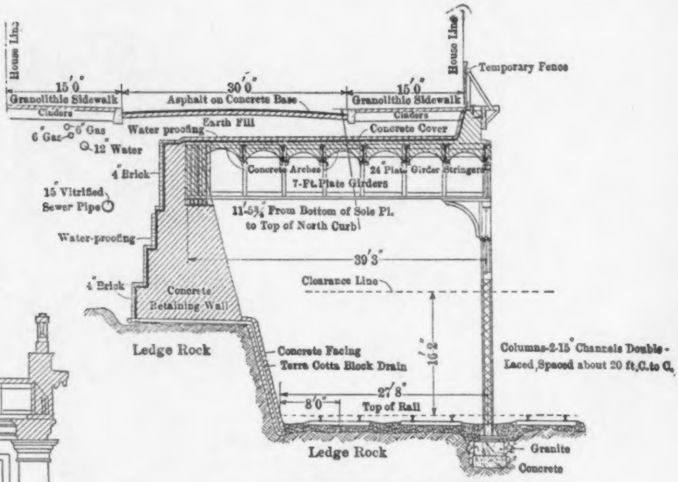
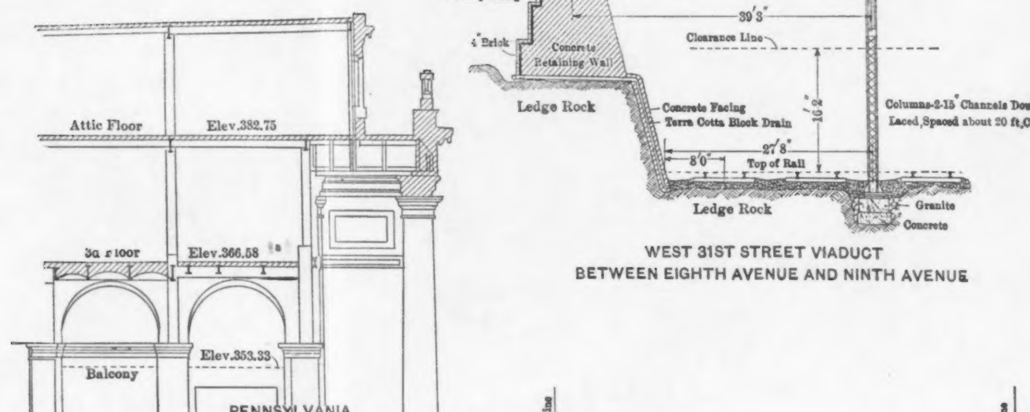


PLATE L.
PAPERS, AM. SOC. C. E.
MAY, 1911.
FRANCIS AND O'BRIEN ON
PENNSYLVANIA R. R. TUNNELS: TERMINAL STRUCTURES.

CAL SECTIONS
VIADUCTS ON
AND WEST 33D STREET



WEST 31ST STREET VIADUCT
BETWEEN EIGHTH AVENUE AND NINTH AVENUE



WEST 33D STREET VIADUCT
BETWEEN SEVENTH AVENUE AND EIGHTH AVENUE

for a length of 247 ft. at the west end and on the pilasters forming part of a concrete facing placed over the ledge beneath the retaining wall for a length of 258 ft. at the east end.

A gallery, 15 ft. wide and 457 ft. long, supported on the viaduct columns on the south and suspended from the roadway deck girders on its northerly margin, forms a part of the entrance concourse, and is connected by stairway to the south sidewalk of 33d Street, and by a northerly extension to the 34th Street temporary entrance and exit.

The excavation to sub-grade, the construction of the retaining wall, the Station Building and the temporary entrance and exit to 34th Street, also the architectural finish of the concourse deck of this viaduct were executed under other contracts.

The excavation for the fifty-six column foundations of this viaduct was started in October, 1906, and completed in September, 1907. The foundations were started in November, 1906, and completed in September, 1907. The steel erection was started in July and completed in December, 1907. The superstructure masonry was started in August and completed in December, 1907. The temporary pavement of the roadway was completed and the highway opened to traffic in March, 1908.

The 31st Street Viaduct, between Eighth and Ninth Avenues, is 800 ft. long and 40 ft. wide. It is a single-deck structure supported on the retaining wall on the south and on steel columns on the north. The columns are placed between the tracks and in the platforms at intervals varying from 19 ft. 5 in. to 43 ft. 1 in. from center to center, and, on account of the track layout, the twelve easterly and five intermediate columns are placed in a line askew to the viaduct and distant 32 ft. 6 in. (maximum) north of the north house line.

The deck framing is made up of riveted longitudinal stringers spaced about 5 ft. 5 in. from center to center, each built up of one 24 by $\frac{3}{8}$-in. web-plate and four 6 by 6 by $\frac{1}{2}$-in. L's, and face-connected at the ends to the webs of the transverse girders, which, in turn, rest at the south end on steel I-beam grillages set in pockets left in the retaining wall for the purpose, and top-connected to the columns or face-connected to the longitudinal girders on the north end. Each of these transverse girders is built up of one web-plate varying from 62 by $\frac{1}{2}$ in. to 96 by $\frac{9}{16}$ in., with four flange L's varying from 6 by 6 by $\frac{5}{8}$-in. to 8 by 8 by $\frac{3}{4}$-in., and from two to four cover-plates 15 and 18 in. wide.

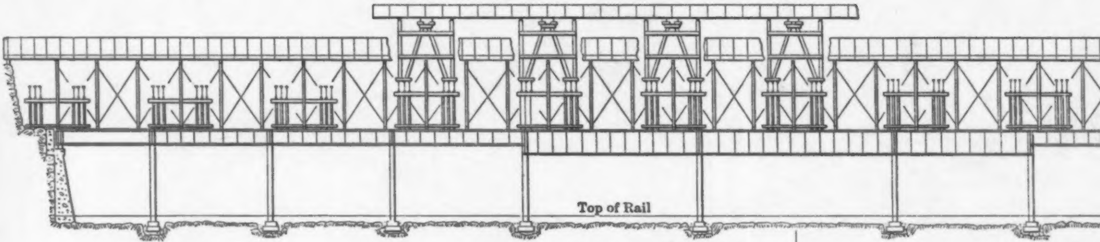
The columns are made of two 15-in. channels, double latticed, and reinforced in some instances by 15-in. cover-plates. They vary in length from 30 ft. 8 in. to 43 ft. 2 in. There are riveted plate and angle brackets at all girder connections to columns. The columns are braced, longitudinally of the viaduct, in pairs, by a battened diagonal bracing system with top and bottom latticed struts, thus making them almost equally strong in each direction.

For a length of about 400 ft. at the east end, the viaduct is opposite the U. S. Post Office Building, and as it has been determined to support certain minor features of the building entrances and areas on this viaduct, the latter has required strengthening.

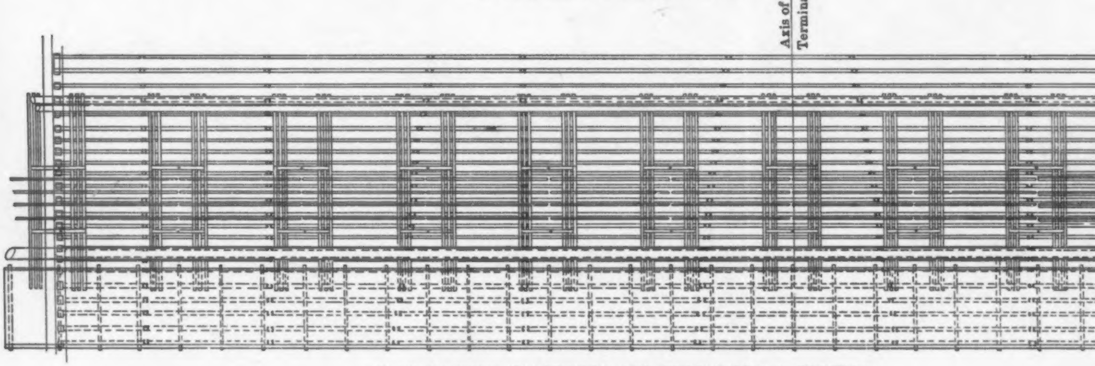
This strengthening has been accomplished by introducing a diagonal bracing system, which is connected to the transverse girders at the top, and to the lowest point of attachment of the original longitudinal column bracing about midway of the column length at the bottom. A lateral system has been introduced in the plane of these diagonals. By the introduction of the diagonal system referred to, the columns were made stronger in a north and south direction than in the opposite direction. Hence, where the difference in value under actual loading conditions required it, the longitudinal bracing was modified and extended down the column sufficiently to compensate.

Between the deck stringers, semicircular spandrel concrete arches were turned, and the deck was water-proofed and completed in the manner hereinbefore described for the 31st Street Viaduct between Seventh and Eighth Avenues, except that the completion of the north sidewalk was a part of the viaduct work and therefore a superimposed concrete retaining wall or bulkhead was built monolithic with the northerly arches on the north house line to within 6 in. of the top of the sidewalk in order to retain the fill. The deck water-proofing is turned up the back of this wall. Tie-rods were provided at intervals of about 7 ft. between the stringers in order to take up the thrust of the deck arches. The contractors requested permission to omit these, on account of the nuisance of constantly readjusting forms, and were permitted to omit all but those in the north panels. A timber frame wood fence was erected on the north house line and attached to the superimposed bulkhead wall.

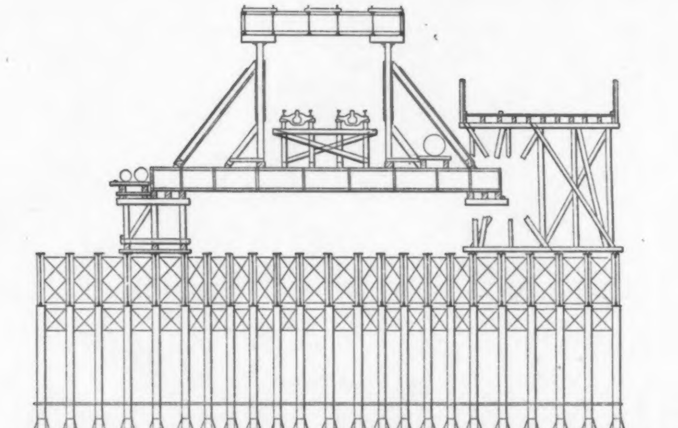
The excavation to sub-grade and the construction of the retaining wall were executed under another contract.



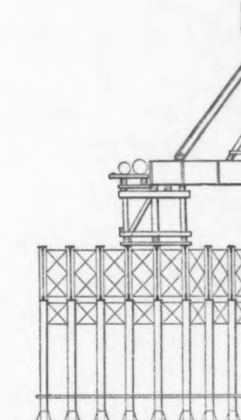
SIDE ELEVATION LOOKING WEST



PLAN SHOWING FALSEWORK AND STEELWORK IN POSITION

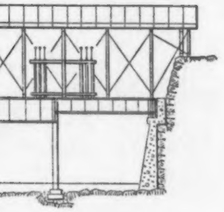


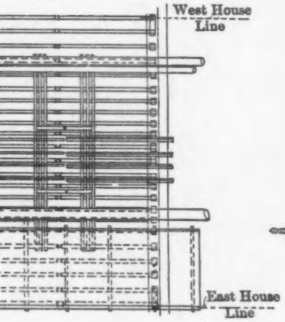
SECTION SHOWING FALSEWORK AND STEELWORK IN POSITION

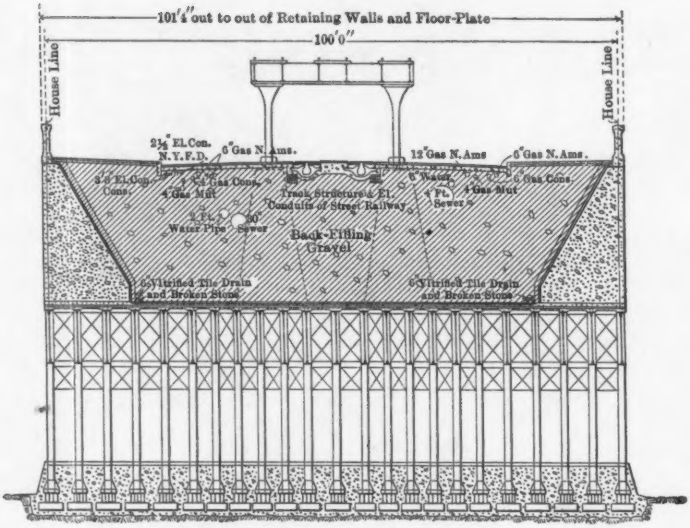


SECTION SHOWING F

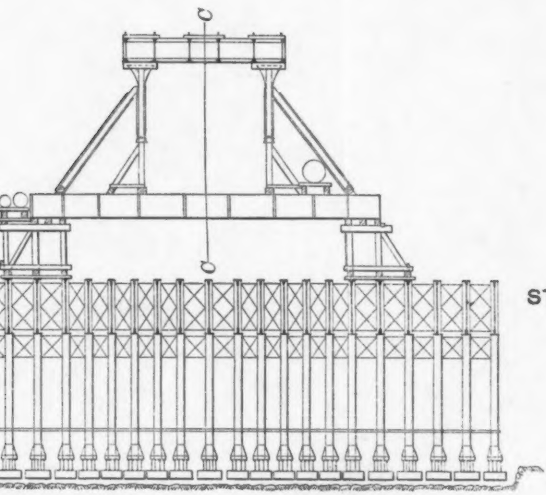
PLATE LI.
PAPERS, AM. SOC. C. E.
MAY, 1911.
FRANCIS AND O'BRIEN ON
PENNSYLVANIA R. R. TUNNELS: TERMINAL STRUCTURES.



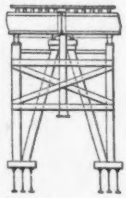




SECTION SHOWING COMPLETED WORK



ION SHOWING FALSEWORK AND STEELWORK IN POSITION



SECTION "C-C"

STEEL VIADUCT UNDER 9TH AVE.

PLAN AND SECTIONS
SHOWING RELATION OF TEMPORARY TO
PERMANENT STRUCTURE

The excavation for the twenty-nine column foundations of this viaduct was started in September, 1907, and completed in July, 1908. The foundations were started in October, 1907, and completed in July, 1908. The steel erection was started in June, 1908, and completed in March, 1909. The deck masonry was started in April, 1908, and completed in April, 1909. The highway was restored to traffic by the completion of the temporary pavement in December, 1909.

The 33d Street Viaduct, between Eighth and Ninth Avenues, is 800 ft. long and 43 ft. wide, and is identical in type with the 31st Street Viaduct between Eighth and Ninth Avenues, except that the columns are all in the same straight line (approximately on the south house line).

The excavation to sub-grade and the construction of the north retaining wall were executed under another contract.

The excavation for the twenty column foundations of this viaduct was started in October, 1907, and completed in December, 1908. The foundations were started in February, 1908, and completed in January, 1909. The steel erection was started in March, 1908, and completed in May, 1909. The deck masonry was started in April, 1908, and completed in June, 1909. The highway was restored to traffic by the completion of the temporary pavement in July, 1909.

The 31st Street Viaduct, west of Ninth Avenue, is 200 ft. long and 40 ft. wide, and is similar in type to the 31st Street Viaduct between Eighth and Ninth Avenues. It is opposite the site of a proposed Express Building, and has been designed so that an intermediate floor connected with such a building may be constructed at some future date.

To provide a suitable depth of story beneath the roadway deck, the transverse girders of the latter are placed in pairs, one on each side of the columns, with riveted diaphragms at the stringer connections. By this arrangement the transverse girders were reduced to 48 in. in depth, but this design resulted in erection difficulties not encountered in the more simple types used elsewhere.

The excavation to sub-grade and the construction of the south retaining wall were executed under another contract.

The excavation for the eight column foundations of this viaduct was started in March and completed in August, 1909. The foundations were started in April and completed in August, 1909. The steel erec-

tion was started in May and completed in September, 1909. The deck masonry was started in August and completed in November, 1909. The highway was restored to traffic by the completion of the temporary pavement in January, 1910.

Loads.—In determining the weight of the structures, for the purpose of calculating the strains, the weight of masonry floors has been assumed at 140 lb. per cu. ft. The decks immediately beneath 31st and 33d Streets have been designed to carry concentrated live loads in accordance with the diagram, Fig. 2, placed so as to give maximum strains in all parts of the structure.

Longitudinal spacing for loads *"A"* 6½ ft. and 24 ft., alternately.
Longitudinal " " " *"B"* 20 ft. apart.
All loads to be placed so as to produce the maximum effect on all parts of the structure.
Add 25% to the live loads indicated by Fig. 2 to compensate for impact and vibration.

LIVE-LOAD DIAGRAM FOR UPPER DECK OF 31ST STREET AND 33D STREET BRIDGES.

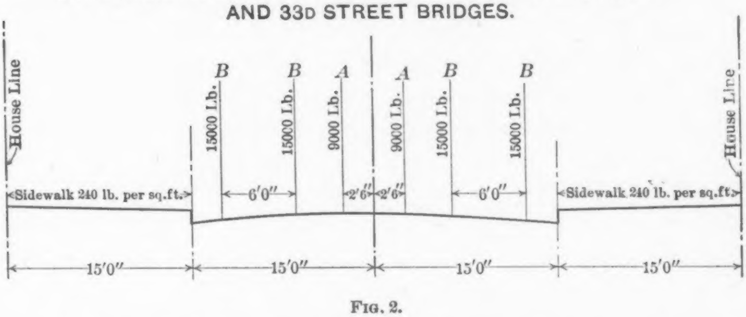


FIG. 2.

In addition to the live loads indicated by Fig. 2, the upper decks of the 31st and 33d Street bridges have been designed to sustain (together with the weight of the structure) a dead load of fill and pavement equal to 135 lb. per cu. ft.

The lower decks of the 31st and 33d Street viaducts between Seventh and Eighth Avenues have been designed to carry a live load of 240 lb. per sq. ft., uniformly distributed.

To compensate for the effect of impact and vibration, 25% of the maximum strains resulting from the above mentioned live loads have been added thereto.

PLATE LII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
FRANCIS AND O'BRIEN ON
PENNSYLVANIA R. R. TUNNELS: TERMINAL STRUCTURES.

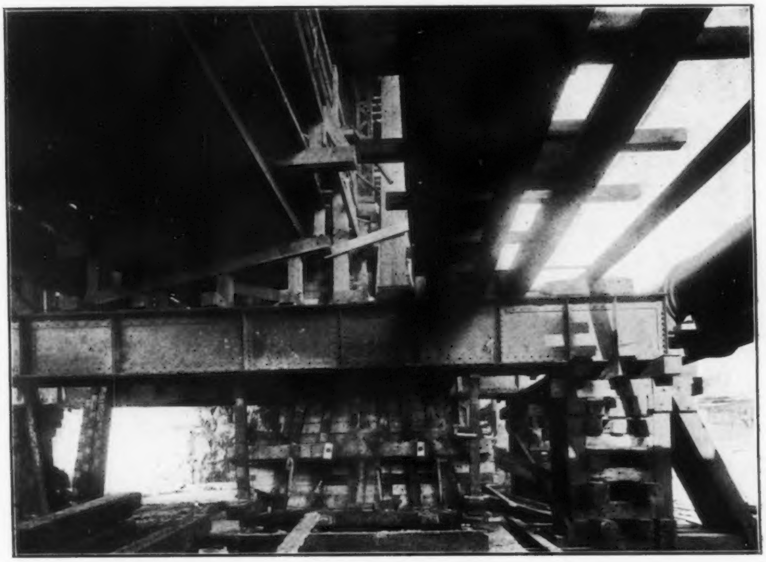


FIG. 1.—FALSEWORKS AND CONCRETE PIER FOR SUPPORT OF ELEVATED RAILWAY, NINTH AVENUE.

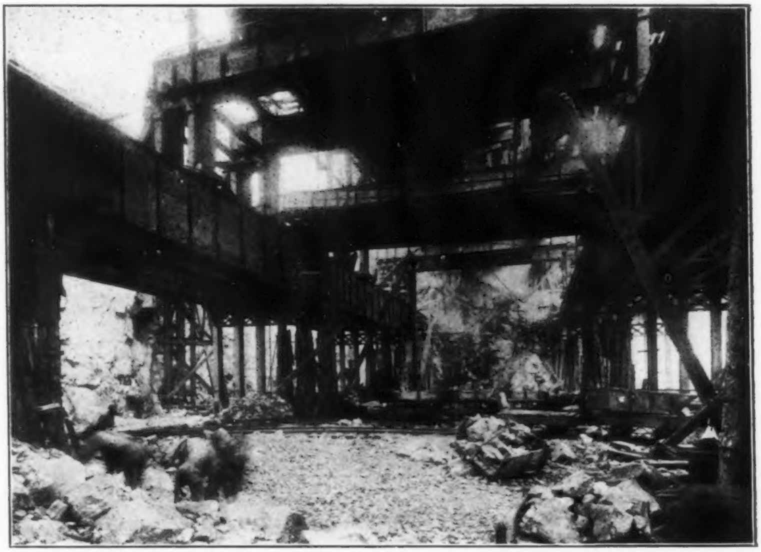


FIG. 2.—NINTH AVENUE BRIDGE DURING CONSTRUCTION.

The Ninth Avenue Viaduct.—The Ninth Avenue Viaduct is 376 ft. long and 100 ft. wide, and extends across the terminal from the south curb line of 31st Street to a point about 110 ft. south of 33d Street. It supports eighteen columns of the elevated railway structure. The franchise required that the minimum depth from the street surface to the top of the structure should be 19 ft. For the reasons hereinbefore stated, it was determined to restore the highway by back-filling over the viaduct roof; hence the latter was designed to support the weight of the back-fill in addition to the weight from the elevated railroad structure, and a uniform live load of 300 lb. per sq. ft. of street surface. The column bents are located wherever practicable between the tracks, but, owing to the relation of the viaduct location to the track layout, the resulting spans vary from about 33 to about 68 ft.

The contract required that the highway and elevated railway traffic should be maintained in service during the construction of the viaduct, hence it became necessary to design the structure so as to simplify its erection. Owing to the comparatively long spans and the heavy loads, it was decided to erect a top-bearing, plate-girder structure, all girders to be set longitudinally, and spaced 5 ft. from center to center, except under the foundations for the elevated railway columns, where they were to be spaced 4 ft. from center to center. The columns in all bents were to be similarly spaced.

The extreme north and south ends of the structure rest on abutment-wall bridge seats. Lateral stability is provided by introducing a horizontal lateral system between the top flanges of each marginal pair of girders, also by introducing substantial cross-frames between the girders and at the tops of the columns.

The floor-plate is made up of 6-in., 12¼-lb. I's, laid transversely, and 6 in. from center to center under the superimposed retaining walls and elevated railway columns, and 8 in. from center to center elsewhere, spanning at least three girders, and with broken joints. These beams are encased in concrete, the floor-plate being similar to that in the "Easterly Portion," hereinbefore described.

The foundation piers, of 1:2:4 Portland cement concrete, 5 ft. square at the top and 12 ft. 6 in. square at the base, are built as monoliths, from the top of the floor-plate to the base of the elevated railroad columns, just below the surface of the avenue. The resulting load on the structure at the base of these foundations is approximately

4 000 lb. per sq. ft.; the superimposed load elsewhere on the viaduct is about 3 000 lb. per sq. ft.

Superimposed retaining walls extend from the floor-plate to the street level on both faces of the viaduct; the basin thus formed is water-proofed, covered with masonry, and contains the back-filling.

Each viaduct column is built up with two 15-in. double-latticed channels, and from two to eight cover-plates, 15 in. wide. Each is provided with riveted plate and angle diaphragms at top and bottom, cap-plates, and riveted plate and angle knee-brackets at top, connected to girders and riveted plate and fitted stiffener angle bases; also clip angles for the attachment of the fender curb. The columns with least loads bear directly on cut granite cap-stones, underpinned to solid rock by 1 : 2½ : 5 Portland cement concrete piers. The more heavily loaded columns bear on steel I-beam grillages, underpinned to solid rock by 1 : 2½ : 5 Portland cement concrete piers. The column bases are secured to the granite cap-stones by two 1¼-in. fox-bolts, 12 in. long, and to the grillages by four ⅞-in. bolts in each column base.

The viaduct girders are 6 and 9 ft. deep, and each is built up of one web-plate with four side-plates, where required, four 6 by 6-in. or four 8 by 8-in. flange angles and from two to twelve cover-plates, 16, 18, and 20 in. wide. Where the girders change in depth from 9 to 6 ft., a riveted plate and stiffener angle seat, double-spliced to the webs and milled to fit the bottom flange angles, is provided on the ends of the 9-ft. girders over the column bearings. Connecting plates are provided on the top flange at one end of each girder and are riveted up to the abutting girder.

On the abutment walls the girders bear on cast-iron pedestals, or steel channel bolsters with top and bottom riveted plates, which, in turn, rest on the cut granite bridge seat.

The scheme for the support of the highway and the elevated railway structure, during the erection of the viaduct, required that a central core of rock be left in place, longitudinally, until the construction of the east and west margins of the permanent viaduct, for a width of about 28 ft. on each side, could be completed; hence the permanent bridging was erected on the east and west margins of the structure, beginning at the north end, and the vertical supports for the falsework were transferred to this permanent bridging. Then the excavation was done, after which the intermediate section was

constructed. The falseworks were arranged so that the foundations for the elevated railway columns could be completed without disturbing them, and they were removed as rapidly as the completion of the permanent foundations permitted.

The columns are encased in continuous concrete fender-walls, with battered faces, from the base of each column to 4 ft. above the top of rail. A reinforcing mesh is placed within 1 in. of the faces of these fender-walls in order to toughen the surface and reduce the tendency to check. There are refuge openings in the fender-walls at intervals of about 25 ft.

The excavation for the 161 column foundations of this viaduct was started in September, 1908, and completed in July, 1909. The foundations were started in October, 1908, and completed in July, 1909. The steel erection was commenced in January, 1908, and completed in August, 1909. The superstructure masonry was started in November, 1908, and completed in September, 1909. The back-filling for the restoration of the roadway was commenced in April, and completed in December, 1909. The elevated railway was transferred from temporary to permanent foundations from time to time as the latter were completed. These foundations were started in November, 1908, and completed in August, 1909. The highway was restored to traffic by the completion of the temporary pavement in January, 1910.

The specifications for the viaducts did not vary much, in respect to materials and workmanship, from those quoted in Appendix A. Extracts from the specifications for the manufacture of the steel for the viaducts are given in Appendix B.

SUBSTRUCTURES.

All work below the track level between the marginal retaining walls, from the east side of Tenth Avenue to the west side of Seventh Avenue, except the viaduct foundations, was included in the Substructures Contract, the principal features of which were: Pipe and express trucking subways, elevator pits, a complete under-drainage system, a traction conduit system, and the foundations for the Station Building and the U. S. Post Office Building.

Subways.—Due to the fact that the Station Building is superimposed over the tracks in such a manner that complete provision

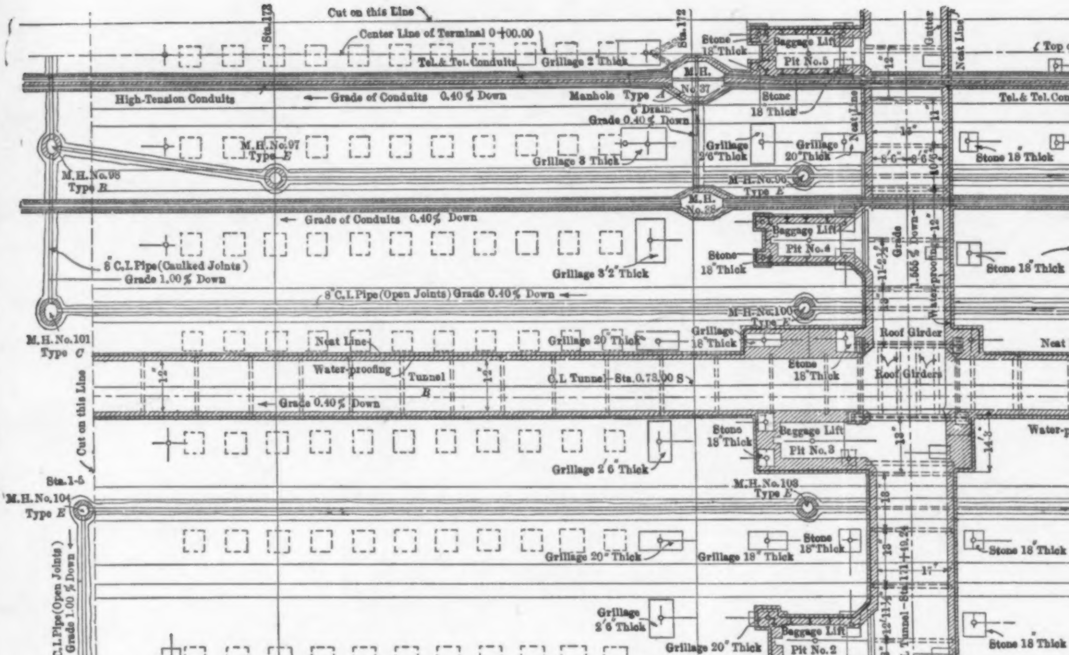
for supply and discharge service lines could not be made advantageously within the building itself, and due to the necessity for the installation of a very extensive yard service system, pipe subways are provided beneath the tracks, running across the yard at intervals of about 400 ft., one of them leading directly from the sub-basement of the Service Building. These cross-subways are built, in some cases, for trucking subways, and are connected to a longitudinal subway extending from Seventh to Ninth Avenues, to provide for the transfer of express matter between the trains and the Express Building site west of Ninth Avenue, and also to provide for the transfer of certain mail matter between the trains and the Post Office site west of Eighth Avenue.

The pipe subways vary in clear width from 4 to 12 ft. and in clear height from 10 to 15½ ft. They contain mains for fire service, water supply, car heating, air-brake testing, the operations of interlocking, the heating of the Station Building, also certain soil pipes and rain-water leaders from the Station Building, drainage lines from the yard buildings, electric conduit feeders for yard, platform, and subway lighting, and for the operation of the electro-pneumatic interlocking plant. These service lines are carried beneath the roof of the trucking subways and above the head-room for trucking.

For the discharge of soil drainage from the yard buildings, and for that portion of the soil drainage from the Station Building which could not be discharged by gravity, ejectors are placed in pits provided for the purpose; these pits are connected to and form a part of the subway system.

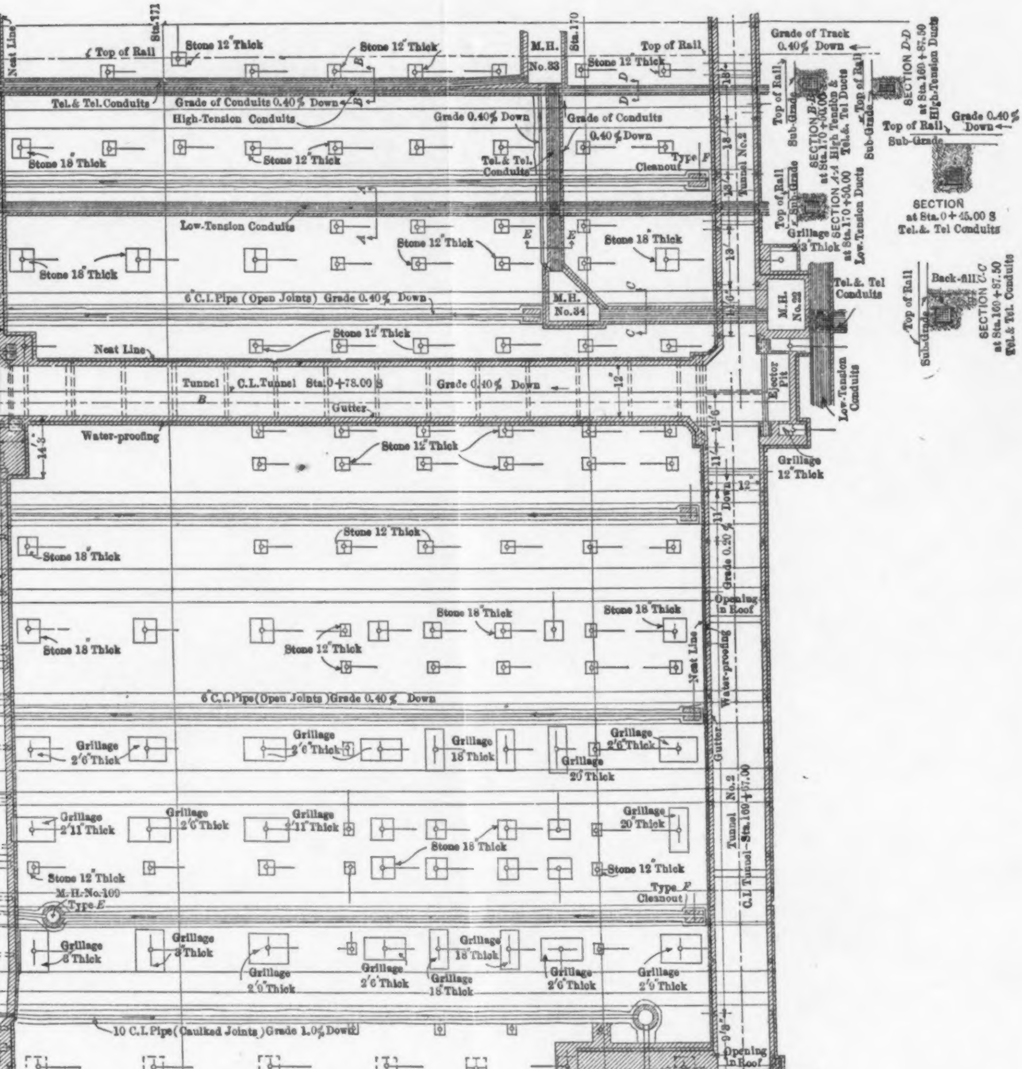
The cross-trucking subways are 17 ft. wide in the clear and have a height of about 12 ft. The longitudinal subways are 12 ft. wide in the clear and about 13 ft. high. Beneath the pipes there is clear head-room of 8 ft. for trucking.

The subways are of reinforced concrete of box section, the roofs and side-walls being designed to support the track bed and train loads, and the whole cross-section to resist hydrostatic pressure, due to an assumed head of water measured below the bottom of the under-drainage system, which is at an average depth of 1 ft. 6 in. below the subway roofs. In order to reduce the superimposed dead load to a minimum, the subway roofs are placed at sub-grade, 2 ft. below top of rail. The train loads were reduced to an equivalent uniformly-dis-



GENERAL LAYOUT, FOR SUB-STRUCTURES

PLATE LIII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
FRANCIS AND O'BRIEN ON
PENNSYLVANIA R. R. TUNNELS: TERMINAL STRUCTURES.



CTURES

tributed live load per square foot of subway roof. Subway roofs beneath platforms are of lighter section, but are strong enough to provide for such loads as may come upon them due to the use of the space below the platforms for branch service lines.

The reinforcement of the subways is of plain, square, merchantable rods, having a permissible unit stress of 16 000 lb. per sq. in., and spaced at intervals of 6 in. from center to center. Similar bars were set longitudinally and wired to the transverse bars wherever practicable, for reinforcement against temperature effects and shrinkage. These temperature bars are at intervals of 12 in. from center to center throughout the cross-section, a single system being used in the floors and side-walls and a double system with alternate spacing in the roof. The interior face of the reinforcement is 1½ in. back of the finished interior face of each subway. The subways are completely enveloped with six-ply pitch and felt water-proofing, and protected by masonry. Where conduit banks cross the subways, they are constructed in reinforced concrete troughs just below the subway roofs. The side-walls of these troughs are designed as reinforced concrete girders, and the floor slabs of the troughs are suspended from them. Many pipes pass through the walls of the subways, and in order to provide for putting these in after the subways had been constructed, and to insure a water-tight job on completion, pipe sleeves, made up of pipes of varying diameters, threaded on the outside and each fitted with two cast-iron screw flanges, with lead washers, are set in the walls and clamped to the water-proofing, a sufficient space having been allowed between the external perimeter of the pipes and the internal perimeter of the sleeves to permit of caulking the external joint between them after placing the pipes.

Numerous openings have been left in the roofs of the subways for pipe branches under platforms, and elsewhere, for connection with surface pipe trenches. The presence of foundations, conduit lines, drainage lines, etc., adjacent to and connected with the subway system, made the execution of some portions of the substructure work a very painstaking task.

Pockets have been left about 2 ft. below the roofs of the pipe subways at intervals of 12 ft. from center to center, thus providing bearings for the pipe-hanger beams, each of which is made up of two 6-in., 8-lb. channels, back to back. In the trucking subways, 9-in.,

21-lb. I-beams are built into the roof section, with their bottom flanges flush with the ceilings. Three-piece wedge forms were set up over these I-beam flanges and withdrawn on the completion of the work, thus leaving the flanges free and clear for the attachment of the pipe hangers.

In general, the method of procedure in the construction of the subways was as follows:

A trench was excavated and a concrete floor base was placed, ready for water-proofing; then backing-walls, varying in height from 2 ft. to the full height of the subway, according to the conditions encountered, were built, usually for stretches of 100 ft. at a time. Owing to the nature of the rock and the presence of deep pockets of earth, the trenches were in some cases excavated from 5 to 10 ft. wider than required by the drawings, and in these instances, after experimenting with various heights of backing-wall, it was finally determined to build the backing-walls in the first stage to a height of about 4 ft., thus forming with the floor base a basin, which was then water-proofed. After completing this first stage of water-proofing, the reinforced concrete floor was laid with battered margins arranged to obtain the effect of a flat inverted arch thrusting against the side-walls after the latter were completed. Gutters were provided in the floors at one wall angle, to carry off any water which might accumulate from pipe leaks or otherwise. After the reinforced concrete floors had set, a four-piece collapsible timber form, about 30 ft. long, with suitable lagging on its external perimeter, was set up over the floor and adjusted by wedges and keys to the internal dimensions of the subway. The reinforcement was placed at the proper distance from this form, and secured in place by suspending it from scantlings blocked up on the roof of the form. The side-wall reinforcement was bent at the bottom to conform to the above described battered faces of the floor. The outside forms for the walls were constructed separately on the backing-walls above described, and were braced to the sides of the trenches, after which the side-wall and roof concrete was placed as a monolith. The longitudinal reinforcement was extended beyond each section, and lapped for continuity. After the tube concrete had set sufficiently, the external forms were removed, the water-proofing was completed, and the masonry protection was placed. After a sufficient time had elapsed to insure the thorough set of the side-walls

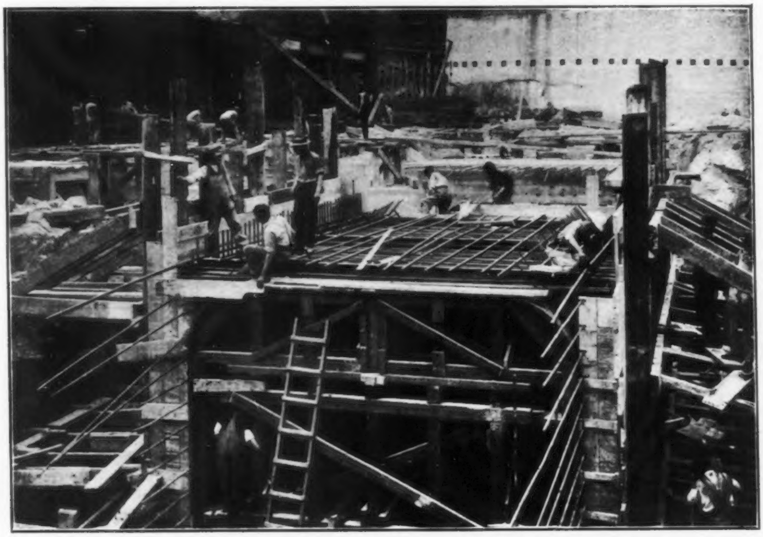


FIG. 1.—TRUCKING SUBWAY DURING CONSTRUCTION.

FIG. 2.—TRUCKING SUBWAY DURING CONSTRUCTION.

and roof, the interior form was pushed ahead and made ready for the next section.

Owing to the great amount of time consumed in placing concrete backing-walls in open trenches, the contractors were permitted to substitute terra cotta tile backing above the 4-ft. wall placed in the first stage. A concrete cover 4 in. thick was placed over the roof water-proofing throughout.

Elevator Pits.—Elevator pits have been provided for the baggage, mail, and express lifts. Nine are connected directly to the main cross-trucking subway just east of Eighth Avenue, two are connected with that subway by a branch at its north end, and two others, just west of Seventh Avenue on the north side of the terminal, are inter-connected by a special trucking subway with a branch connection to the easterly pipe subway.

These lift pits are open at one end and at the top, hence a steel stud and concrete wall structure with reinforced concrete floor was adopted. The floors of the pits are about 5 ft. below the floors of the trucking subway. The pits are 8 ft. wide, 15 ft. 4 in. long, and vary in height from about 18 ft. to 20 ft. The floors and walls are water-proofed with six-ply pitch and felt. The lifts are of the standard plunger type, and the plunger castings are made up of stud rings and clamping rings; to insure tight work, the floor water-proofing was cut to fit the studs on these castings and clamped in place.

The reinforcement of the floors is of $\frac{3}{4}$-in. plain, square, merchantable rods, laid transversely at intervals of 6 in. from center to center, with special framing around the plunger castings. This floor was designed to resist the hydrostatic pressure due to a head of water measured from the bottom of the under-drainage system. The top surface of each pit floor has been graded to a semicircular sump at one end of the pit and on its axis. This sump has a radius of 12 in. and is 6 in. deep. The wall framing is of 15-in. I-beams set vertically, spaced at intervals of about 4 ft. 6 in. from center to center, connected at the top with 15-in. I-beam curbs, and provided with clip angles at the bottom for anchorage. The side-walls of the pits are of concrete, 18 in. thick, finished flush on the inside with flanges of steel studs, water-proofed on the outside, and backed up with masonry.

Chases 18 in. square have been left in the end-walls of the pits for pressure pipes; these walls are normally 3 ft. thick.

Many of the pits are located between the Station Building columns, so that, in some instances two, and in other instances four, of these columns are founded below the floors of the pits and encased in their walls. The entrances to the pits are flared at 45°, these flares measuring 4 ft. on the side; and riveted plate-girder lintels are built into the subway roof to support it across these flared openings.

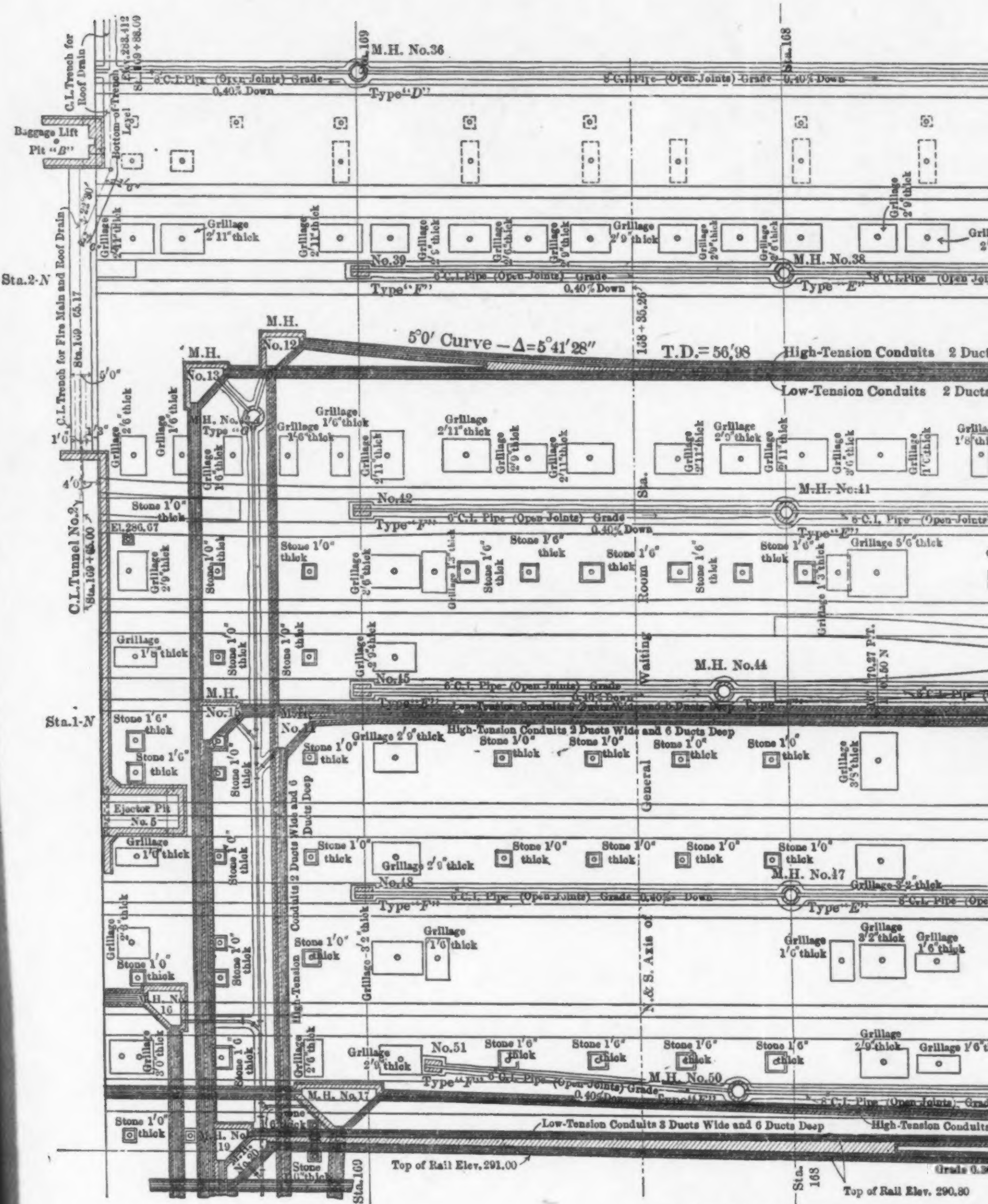
The presence of the Station Building column foundations and the necessity for setting these columns as a part of the lift pit structures made the construction of the latter quite complicated in many instances. To avoid too much delay in the construction of the pits, the Station Building columns in question were spliced just above subgrade, and the lower sections were manufactured and delivered to the substructures contractors and erected by them, thus facilitating progress in the construction of the pits.

In general, the construction of the lift pits proceeded about as follows:

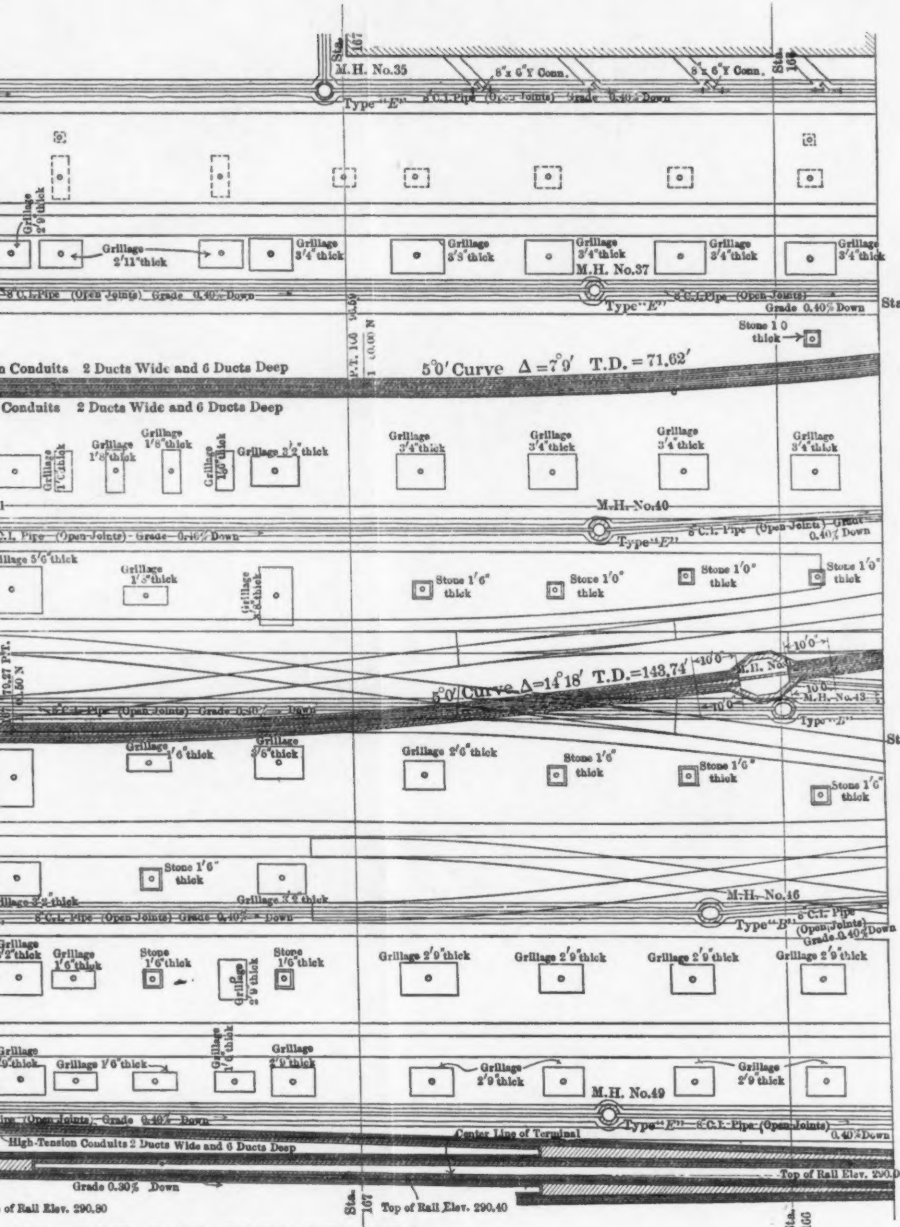
The pits were excavated; a base course of concrete was laid; the concrete facing was carried up against the face of the ledge where the pits were wholly in solid rock, or backing-walls were constructed where the pits were partly in earth, to about the level of the adjoining subway floor base. The basin thus formed was water-proofed, after which, the Station Building column foundations were set, and the lower lengths of these columns were erected. A concrete base, of the width of the side-walls, was placed and on it the side-wall studding was erected. Then the reinforced concrete floor was immediately completed; after which, the side-wall masonry was placed, the water-proofing completed on the outside wherever practicable, and backed up with masonry. In some instances it was necessary to complete the backing-walls and water-proofing before the side-wall studding was erected.

Under-Drainage System.—The summit of the track grades in the terminal area is about 10 ft. below mean high water and about 35 ft. below the general sewer level, and a considerable portion of the terminal yard west of Eighth Avenue is open to the weather.

An elaborate under-drainage system is provided, that portion under the covered area being designed to dispose of seepage from beneath



GENERAL LAYOUT, FOR



...OUT, FOR SUB-STRUCTURES

the enclosing retaining walls, from fissures in the ledge, water from springs, and drip from equipment and platform surfaces. An estimate of the probable maximum quantity of water from these sources, based on experience with somewhat similar conditions, formed the basis of the design for that portion of the drainage system under the covered area.

Open-joint, cast-iron, drain pipes, varying from 6 to 12 in. in diameter, are laid in concrete cradles longitudinally of the terminal at intervals of about 50 ft. from center to center. The tops of these pipes are about 3 ft. below the top of rail. The open-joint drain trenches are back-filled to sub-grade with clean broken stone to insure the proper leaching of surface-water.

The subway system hereinbefore described divides the terminal area into drainage zones, and the longitudinal drain lines are intercepted by caulked-joint collector drains adjacent to these subways, which, in turn, are extended to sumps, whence the water is pumped to the city sewers. The caulked-joint collector drains vary in diameter from 8 to 24 in. They are also laid in concrete cradles, and at sufficient depth to permit them to pass under the conduit lines and in some cases under the subways.

The longitudinal drainage system is laid at approximately the same rate of grade as the track, and sloped east and west from the summit of grades about 500 ft. west of Seventh Avenue. At the point of beginning of each longitudinal drain line, a Y-branch, embedded in concrete and provided with a removable cap, forms a clean-out. There are intermediate clean-out manholes at intervals of about 200 ft. on these drain lines, and manholes on all collector drain lines at the intersections of the longitudinal drains. Such of the latter manholes as are adjacent to sumps have chambers about 3 ft. deep below the lowest outlet, and are known as sump manholes, the purpose of the deepened chamber being to permit of the accumulation and removal of silt, in so far as practicable, outside of the sumps. All drainage manholes are of New York City standard sewer manhole type, with granolithic top-dressed concrete floors, 8-in. brick walls, galvanized-iron ladder rungs, and cast-iron heads and covers. There are Y-branches in the drain lines directly in front of the enclosing retaining walls, at intervals of 25 ft. These Y-branches are connected directly to weep-holes through the walls wherever practicable. The entire zone

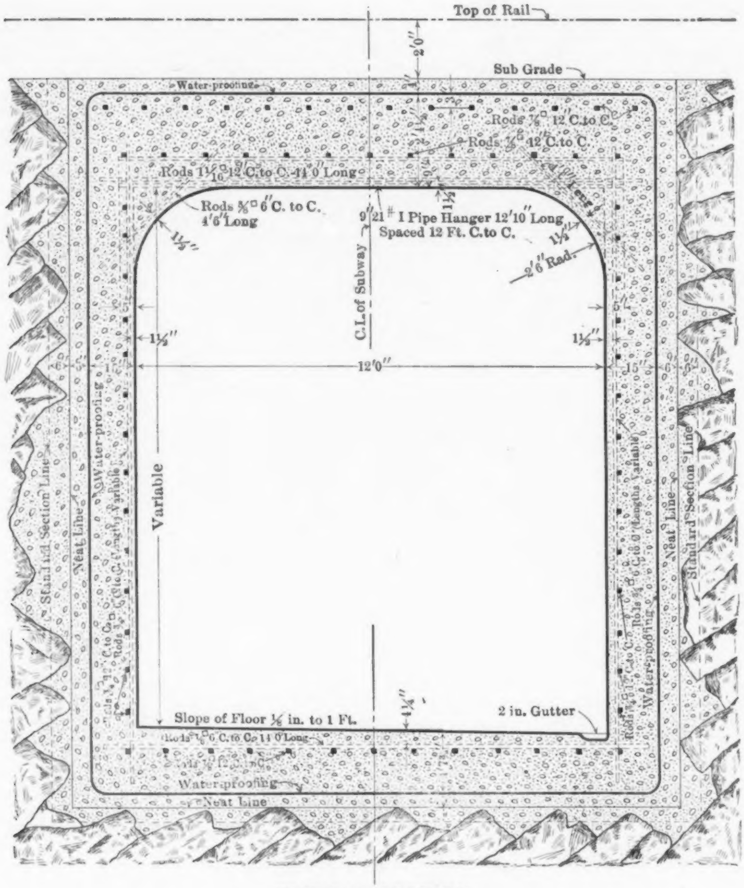
between the summit of track grade and the easterly end of the Easterly Portion contract drains into a sump located just west of Seventh Avenue and south of the 32d Street approach. This sump is a two-story structure, the bottom of which is 22 ft. 6 in. below sub-grade, its roof being at track level. The sump chamber is 23 by 18 by 10 ft., and above this is the machinery room, 23 by 18 by 12 ft. Provision has been made for electrically-operated, centrifugal pumps to be set up on the floor of the machinery-room. This room is connected by a door with the adjacent pipe subway. The suction pits are in the floor of the sump chamber. This sump discharges through a 20-in. pipe into a private sewer built under the west sidewalk of Seventh Avenue, which, in turn, connects with the city sewer at Seventh Avenue and 31st Street.

A zone, immediately west of the track grade summit and south of the longitudinal trucking subway, discharges into a sump having a capacity of 1 000 gal. per min., built beneath the floor of the sub-basement under the south sidewalk of 31st Street in front of the Service Building. This sump has electrically-driven, centrifugal pumps which discharge through an 8-in. pipe into the 31st Street sewer. The remaining area of the westerly slope east of Ninth Avenue empties into a sump, having a capacity of 14 500 gal. per min., just east of Ninth Avenue and south of 33d Street. This sump is a two-story structure, the floor of the chamber being about 28 ft. 6 in. below sub-grade, and the top of the roof about flush with the top of rail.

The sump chamber is L-shaped, the main dimensions being 48 by 22 by 11 ft., and over this is located the machinery-room, the main dimensions of which are 48 by 22 by 13½ ft.

Provision has been made for three electrically-driven, centrifugal pumps (two of which have been installed) with direct-connected motors, the latter being set up on the floor of the machinery-room. This room is connected by a door with the adjacent pipe subway. The suction pits are in the floor of the sump chamber. This sump discharges through a 30-in. pipe into the Ninth Avenue sewer.

The entire westerly slope west of the east line of Ninth Avenue drains into a sump having a capacity of 8 200 gal. per min., which is just east of Tenth Avenue and south of the tunnel portal. This sump is a two-story structure, the floor of the chamber being about 25 ft. below sub-grade, and the top of the roof about flush with the top of



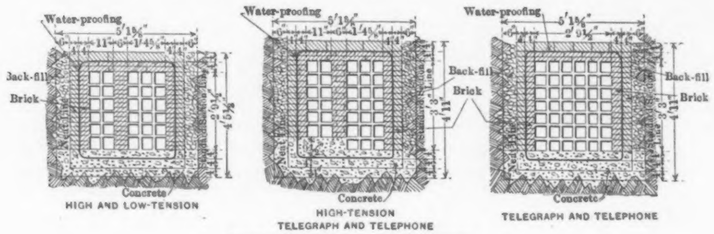
TYPICAL SECTION
OF 12-FT. TRUCKING AND PIPE SUBWAY

Note: In some cases water-proofing was protected on outside of side walls by Terra Cotta tiles instead of concrete as shown.

FIG. 3.

rail. The sump chamber is 40 by 11 by 10½ ft., and over this there is an L-shaped machinery-room, the main dimensions being 40 by 11 by 14 ft. Provision has been made for electrically-operated, centrifugal pumps, to be set up on the floor of the machinery-room. The suction pits are in the floor of the sump chamber. This sump discharges through a 24-in. pipe into the Tenth Avenue sewer.

The sump pit floors throughout are of concrete. The side-walls of the Seventh Avenue and Service Building sumps are of concrete and have a gravity section. The side-walls of the Ninth and Tenth Avenue sumps are of steel studs with concrete between. The intermediate floors and roofs of all sumps are of steel beam framing, with reinforced concrete slabs and concrete beam haunches. The walls of the sumps above the floor level of the machinery-rooms, and the roofs of the sumps, are made water-tight with pitch and felt water-proofing



TYPICAL CONDUIT SECTIONS.

FIG. 4.

protected by masonry. There are vault lights in a portion of the roof of the Ninth Avenue sump to afford light for the machinery-room. There are staircases in the Seventh, Ninth, and Tenth Avenue sumps for access to sub-grade, and the staircase openings in the Ninth and Tenth Avenue sumps are protected by reinforced concrete hoods, the roofs of which are covered with vault lights. Trolley hanger-beams in the machinery-rooms of all sumps facilitate the erection or replacement of machinery, and in the roofs there are hatches, with water-tight covers, for the same purpose.

Conduits.—Two low-tension conduit banks, three ducts wide and six ducts deep, and two high-tension conduit banks, two ducts wide and six ducts deep, are laid in the walls of the tunnels under 32d and 33d Streets east of the Easterly Portion contract, one bank of each class being laid in each tunnel wall, making eight duct banks in all.

FIG. 1.—ELECTRIC CONDUITS DURING CONSTRUCTION.

FIG. 2.—ELECTRIC CONDUITS DURING CONSTRUCTION.

These are continued westward below the terminal sub-grade to points opposite the Service Building, where they are turned southward and connected with splicing chambers in front of the Service Building sub-station.

A low-tension conduit bank, four ducts wide and six ducts deep, and a high-tension conduit bank, two ducts wide and six ducts deep. are laid in the north and south walls of the twin tunnels from the North River, making four banks in all. These are connected through splicing chambers at the tunnel portal, and thence extended eastward below the terminal sub-grade to a point opposite the Service Building, where they are turned southward to splicing chambers in front of the Service Building sub-station.

Two banks of telephone and telegraph ducts, each three ducts wide and seven ducts deep, are extended on either side of the core-wall of the North River tunnel portal from a splicing chamber just east of Tenth Avenue eastward below the terminal sub-grade to a point opposite the Service Building, where they are distributed through splicing chambers which connect through shafts, built in the station platforms, with the pipe gallery beneath the Station Building. Connections have also been made through certain splicing chambers to service lines in Eighth and Ninth Avenues. There are splicing chambers on all conduit lines at intervals not greater than 400 ft., and otherwise where the lines change direction.

The ducts are laid in trenches excavated for the most part in solid rock, the tops of the completed banks being about 1 ft. below sub-grade. The duct banks are encased in brickwork above a concrete base and completely enveloped in a water-proofing of four-ply pitch and felt, which, in turn, is protected by 4 in. of brickwork.

The original design required that the ducts should be encased in concrete and that all water-proofing protection should be of concrete; but, as it was feared that the act of depositing the concrete encasement would distort the duct bank alignment, and force open the joints, causing plugging of the ducts, it was determined to substitute brickwork, so that the entire masonry of the ducts above the concrete base could be executed by skilled labor. The very satisfactory result obtained justified the change in plan. The splicing chambers have concrete floors, brick side-walls, with brick partition walls in the double chambers, and roofs of steel beams and concrete. They

have manhole heads of cast-iron, with extension tops where required, and double covers. The splicing chambers are placed between the tracks for convenient access. They are water-proofed with four-ply pitch and felt, which is protected by 4 in. of brickwork on walls and 4 in. of concrete on roofs. The floors of these manholes are drained by pipe connections to the under-drainage system.

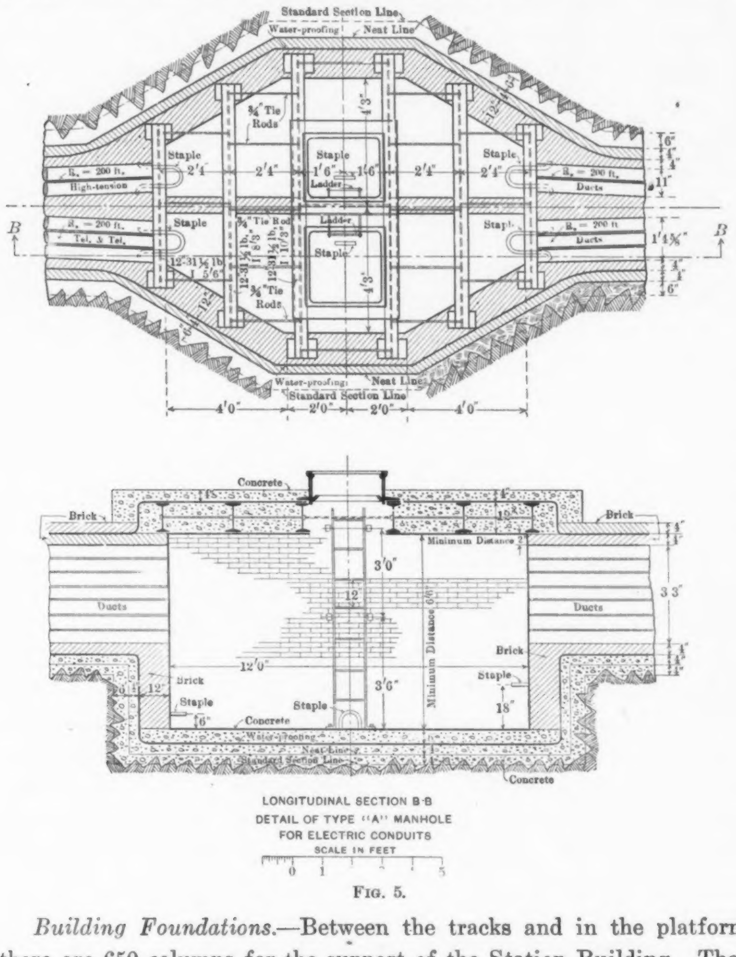


FIG. 5.

Building Foundations.—Between the tracks and in the platforms there are 650 columns for the support of the Station Building. These columns bear on cut granite templates or steel grillages of one or more tiers, underpinned to solid rock by concrete piers varying in

FIG. 1.—ELECTRIC CONDUITS, PIPE SUBWAY, THIRTY-FIRST STREET BRIDGE, AND SERVICE BUILDING, DURING CONSTRUCTION.

FIG. 2.—PIPE SUBWAY AND FOUNDATIONS DURING CONSTRUCTION.

dimensions from 2 by 2½ ft. to 10½ by 14 ft. The column loads vary from 10.5 to 1 658 tons. Grillage bearings were substituted for cut granite cap-stones where the loads prohibited an economical use of the latter. Many columns are located so that their foundations form an integral part of the other substructures, as in the case of columns surrounding elevator pits, hereinbefore described, and columns at junctions of pipe subways. Many other columns are adjacent to pipe and trucking subways, conduit lines, splicing chambers, manholes, etc., the excavation for which, on account of the nature of the rock encountered, required that column foundations be excavated at least to the depth of the excavation for the adjacent structures, in some cases 20 ft. or more below sub-grade.

The 200 columns for the support of the U. S. Post Office Building, west of Eighth Avenue are located between the tracks and in the platforms, and bear on cut granite cap-stones and steel grillages which are underpinned to solid rock by concrete piers. These foundations are similar in character to those for the Station Building. They vary in dimensions from 2 ft. 2 in. square to 7½ by 13 ft. The excavation for many of these was carried to the bottoms of the trenches of adjacent substructures. The specifications for the substructures, in respect to materials and workmanship, do not vary much from those quoted in Appendix A.

Station Building Steelwork.

The design, fabrication, and erection of the structural steel for the Station Building was one of the most difficult problems of the entire terminal. The work of design was begun in 1902, and the last shop cards were checked in December, 1908. Steel deliveries began in May, 1907, and were completed in April, 1909. Erection began about May, 1907, and was substantially completed in September, 1909.

The first contractors for the manufacture of this steelwork went into the hands of receivers in June, 1907, after delivering about 500 tons of material. They had prepared 7 877 shop drawings, nearly all of which had been checked by the engineers. The receivers completed the fabrication and delivery of about 1 800 tons additional. The manufacture and delivery of the remainder of the material was re-let to another firm of contractors who found it necessary, in order to conform to their shop practice, to make 4 719 new shop drawings for their

work: thus almost 13 000 shop drawings were prepared for the Station Building steelwork, and nearly all of these were checked by the engineers.

The engineers, in making the general design from which the shop drawings were prepared, were for several years in constant consultation with the architects, the committees and representatives of the railroad, and the engineers in charge of lighting, heating, ventilating, and other service details, in order that the steel might be designed to fit the architectural and service requirements, and at the same time conform to the prescribed clearances and permissible column locations at track level. During the period of manufacture many important changes were made in the primary arrangement of certain tracks and features of the building, and these introduced some confusion in the readjustment of drawings already made and occasionally of steel already rolled or fabricated. The drafting organization, however, finally completed all revisions, and the material was properly fabricated and erected.

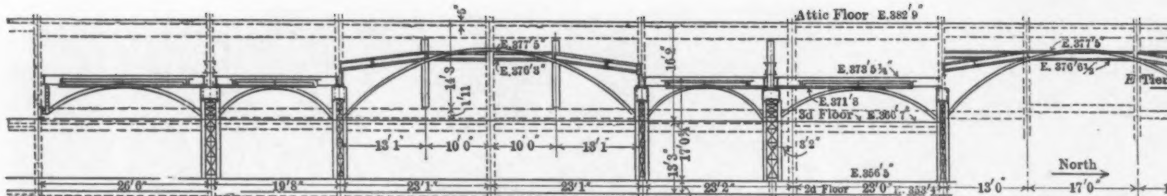
The Station Building covers the site between Seventh and Eighth Avenues and between West 31st and West 33d Streets. It is about 775 ft. long and about 430 ft. wide. It is set back about 19 ft. from the house line of Seventh Avenue, 13 ft. from the house lines of 31st and 33d Streets, and 7 ft. from the house line of Eighth Avenue, thus affording wider sidewalks than are customary. The façades on the streets and avenues rise to a height of about 75 ft. above the curb at the entrance pavilions, and elsewhere to an average height of about 65 ft. above the curb. The portion of the structure enclosing the general waiting-room, the external dimensions of which are 325 by 119 ft., rises in the form of a great lantern 87 ft. above the portions fronting on the streets.

The cubical contents of the Station Building, measured from the track level to the average roof level, is about 40 000 000 cu. ft.

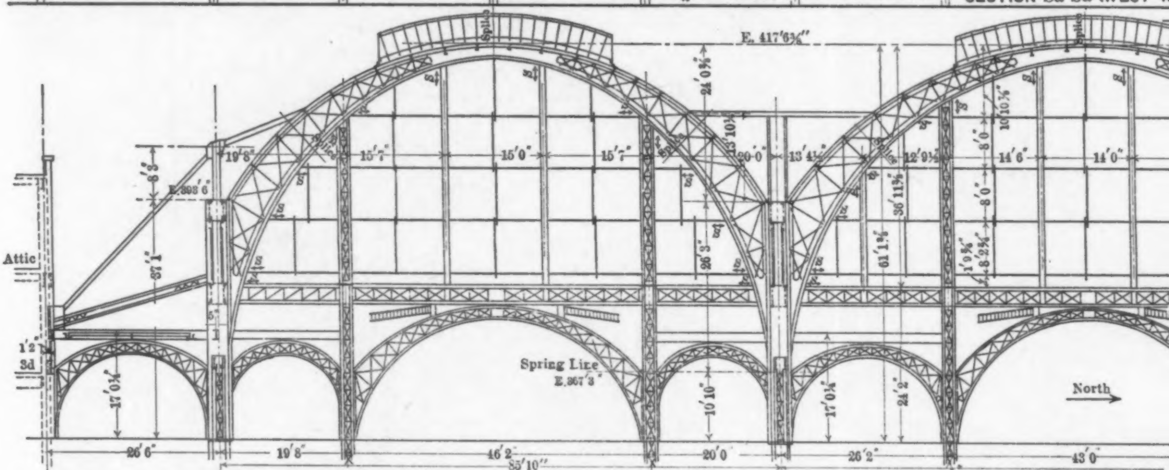
The exterior walls of the building are of cut granite, in solid and ashlar formation, the ashlar being backed up with brickwork.

All interior walls are of brick with face-brick finish, or of brickwork covered with plaster, artificial stone, marble, granite, or combinations of these facings, according to the architectural treatment.

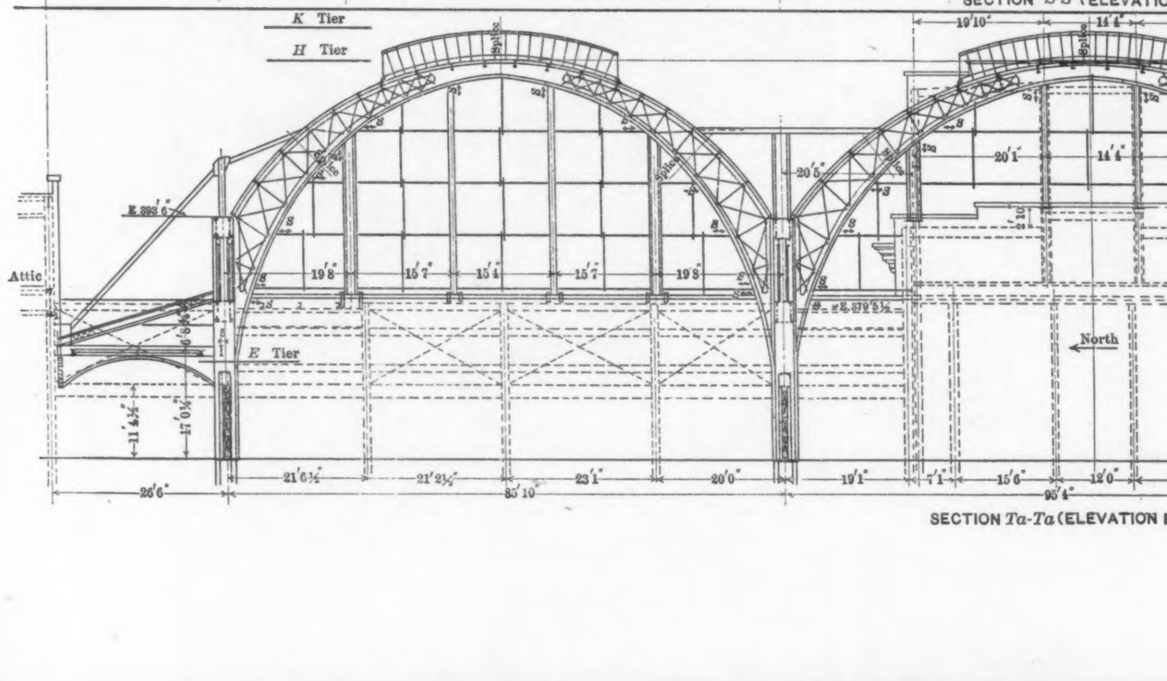
Reinforced concrete floor and roof slabs, with beam haunches of concrete, have been used throughout, except on pitched roofs, where book tiles were laid.



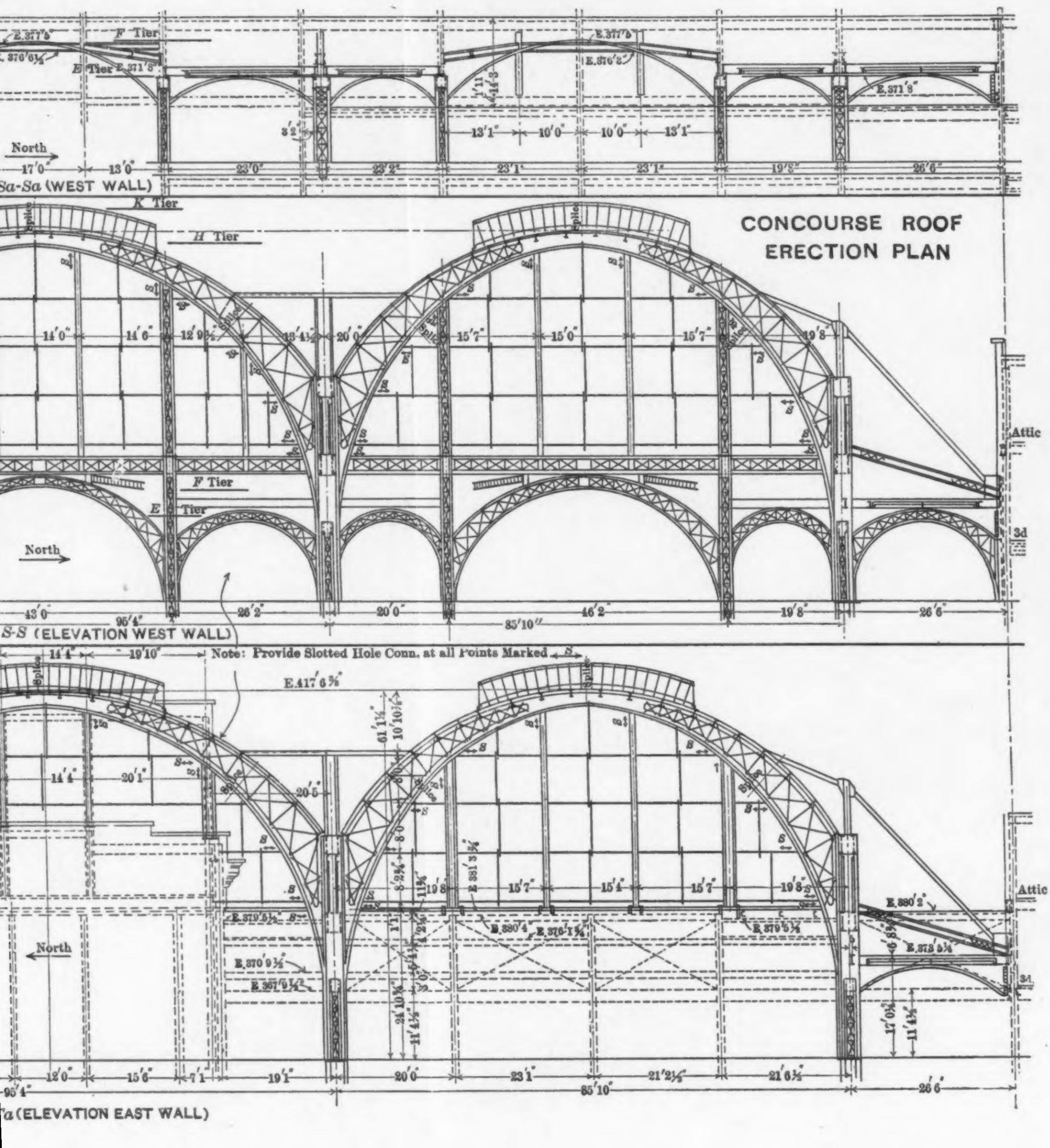
SECTION *Sa-Sa* (WEST W



SECTION *S-S* (ELEVATIO



SECTION *Ta-Ta* (ELEVATION


CONCOURSE ROOF
ERECTION PLAN
Sa-Sa (WEST WALL)
F Tier
E Tier
K Tier
H Tier
North
S-S (ELEVATION WEST WALL)
Note: Provide Slotted Hole Conn. at all Points Marked S
Attic
3d
Ta (ELEVATION EAST WALL)

The total estimated weight of the building, adding all live and dead loads together, is 463 158 700 lb., or 231 579 tons.

The building is superimposed over the tracks, which are spaced so that the columns could be placed between them. Owing to the nature of the problem as a whole, but owing particularly to the relation of the building to the track layout, it was determined to adopt, in so far as practicable, the "cage" type of steel structure, with supporting columns founded at or below track level.

The column locations at track level were fixed by the track layout and clearances, which were determined in advance of the preparation of drawings for the Station Building; and, in providing for the support of the building walls above, unusual girder and offset column arrangements were required in many places.

Outline drawings of the building were prepared by the architects and submitted to the engineers, for preliminary determination of column locations, floor gradients below street levels, sidewalk gradients, types of frames for the various sections of the building, stability of cornices, and other details.

The exact dimensions and character of the building masonry were determined by the architects from time to time after the engineers' preliminary structural studies of the various sections of the building were completed. Subsequently, the working drawings of the steel were prepared ready for the manufacturer.

An examination of the outline drawings first submitted by the architects disclosed the fact that the station was made up of many large inter-connected buildings, more or less structurally dissimilar; hence, for convenience and to facilitate progress in design, the problem was separated into its several component parts, namely: the Seventh Avenue Building, the 31st Street Building, the Eighth Avenue Building, the 33d Street Building, the Easterly Train Sheds, the Arcade, the Restaurant, the General Waiting-Room, the Sub-Waiting-Rooms, and the Concourse. These subdivisions were separately assigned to assistants, each in charge of several draftsmen, and the preliminary studies, final designs, and working drawings were prepared, several subdivisions being advanced at one time, and properly correlated.

It was determined that the tier plans should be made on a scale of $\frac{1}{8}$ in. per ft., which was the scale adopted for the working drawings of tracks and all other structures, the purpose being to have all

constructive features of the terminal laid out on a scale which would permit of ready comparison with the track layout. The scale adopted for tier plans was also adopted for sections and elevations. Typical details of columns, girders, trusses, and important connections were prepared on a scale of ½ in. per ft.

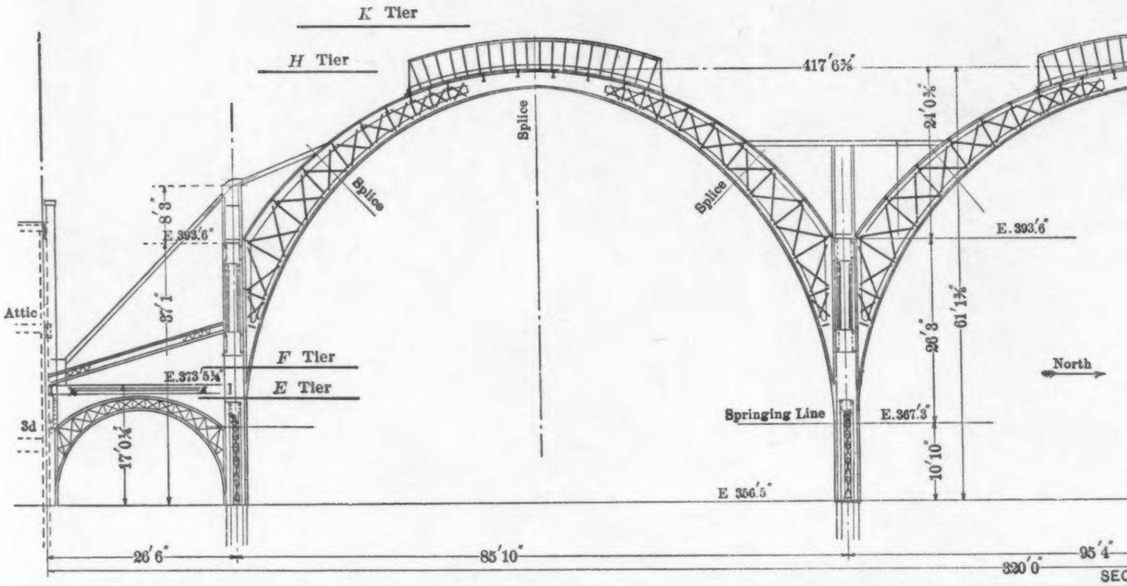
As the preliminary studies advanced, a scheme was laid down, and key diagrams were prepared for the guidance of the draftsmen in making the working drawings. These diagrams showed the various tiers, up to the ceiling of the lantern, divided into four equal parts. Each of these parts was numbered in brown ink prominently on its plan for ready identification. The sheets required to illustrate one-fourth of a tier plan on a scale of ⅛ in. per ft. were each 36 by 60 in., allowing a border of 1 in. all around. The ceiling and roof tiers of the general waiting-room lantern were divided into two equal parts. One-half of the two tiers was shown on one sheet of the foregoing dimensions.

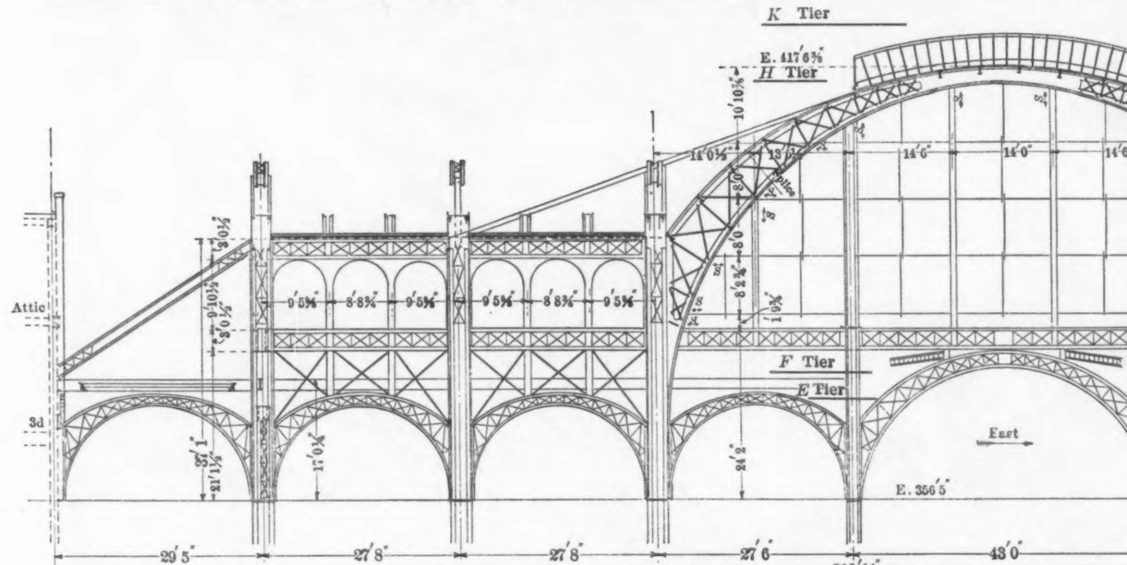
The key diagrams indicate the section lines and their lettering, and these lines and letters are correspondingly shown on all working drawings. To illustrate the scope and intent of the vertical sections, additional key diagrams were prepared on which were indicated the cutting lines of the various tiers, and these were all lettered to correspond with the letters chosen for the tiers.

Brief specifications were prepared for the guidance of the designers, and each responsible assistant was provided with a copy.

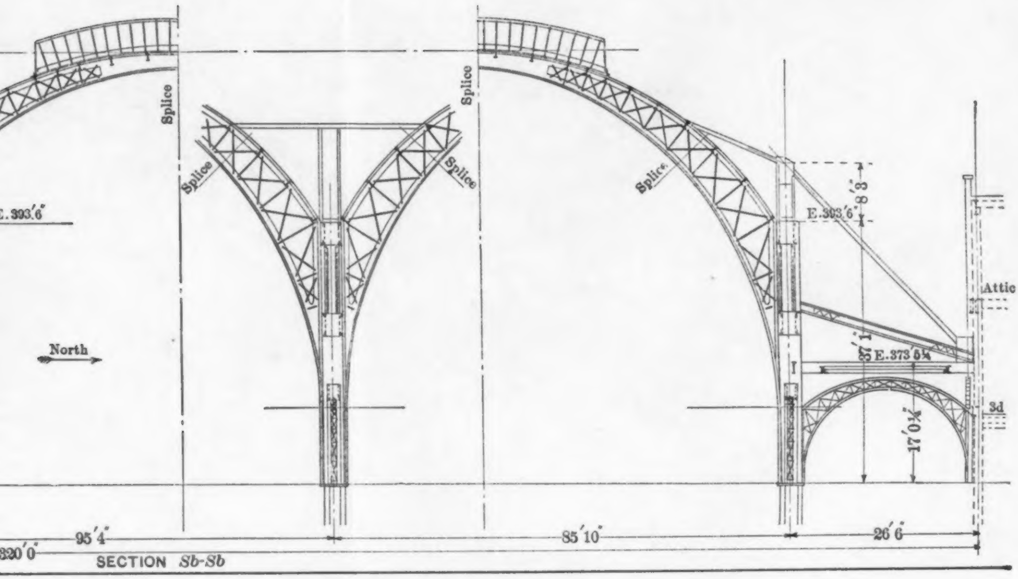
The following system of column numbering was adopted: The east and west lines of columns were numbered from 100 to 2 000, beginning at the south and running north, the ruling column number being on the extreme westerly line, the numbers increasing eastward on the east and west lines. There were 20 main east and west column lines and 35 main north and south column lines, and, by this system, the approximate location of any column was readily determined by its number. Columns off the main lines were given the number of the nearest lined column with compass point suffixes, as 921-*NE*.

Reactions were recorded, in thousands of pounds underscored, at the ends of all girders and trusses, also at the ends of all beams requiring special connections. The sizes of all beams and the materials for all girders were noted against the pieces on the working drawings. The column material was given in the several column schedules. The material for trusses was shown on typical details of the same.





SECTION *MM*
(ELEVATION NORTH WALL)



SECTION *Sb-Sb*

CONCOURSE ROOF ERECTION PLAN

The architectural outline was shown in brown ink on every working drawing.

By agreement with the architects, it was determined, except where the building law governed, that the distance from steel to face of stonework should not be less than 4½ in., and the distance from steel to face of plaster or artificial stone should not be less than 2 in. It was also determined that the distance from the tops of finished sidewalls to the tops of sidewalk beams should be 2½ in., and that the distance from the tops of finished floors to the tops of beams should be 5 in., except for the floors of brick-paved driveways, where the distance from the top of the paving to the tops of the beams was 13 in.

The drawings prepared by the engineers comprised 66 working drawings (tier plans and sections), 7 column schedules, 56 sheets of details, and 40 sheets of studies for revisions and additions, a total of 162 drawings. Copies of these drawings were submitted to the architects for check as to relation of steelwork to masonry and finished wall surfaces before they were issued to the manufacturers.

Working drawings were prepared on record sheets of white mounted drawing paper, with a bending-moment diagram shown thereon against each girder and reaction, recorded in the manner previously stated. All the work was checked independently, and each sheet, with the exception of its strain diagrams, was traced on tracing cloth. The strain diagrams of all trusses and bracing were entered in a record book from which the typical detail sheets of these frames were prepared.

All columns were designed to support the total estimated dead and live loads without reduction. The effect of eccentric loads was provided for, and an effort was made throughout to render the structure stable without dependence on masonry walls. Where granite columns occur, however, the architects elected to omit the steel columns which were originally designed to be encased in them; hence, at such points on the façades, the steelwork stops under the granite columns and bears on top of them.

The design of the ceilings of the carriage driveway entrances, at the corners of 31st and 33d Streets and Seventh Avenue, required that the attic floors be supported partly by suspension from the roofs, and this rather unusual method was carried out.

The Seventh Avenue Building crosses the tracks at the wide end of the fan-shaped easterly approaches, and, owing to the track arrangement, the lower lengths of the columns were spaced at varying intervals longitudinally of the building, and many of them were offset north or south of the superstructure columns. Owing to this offset column arrangement, the spans of the girders supporting the main front wall and granite colonnade vary from 16 ft. 2 in. to 53 ft. 4 in. from center to center of columns, and the girders vary in depth from 3 to 10 ft. Certain of the largest girders are of box-section with two web-plates. The granite columns bear on steel I-beam grillages, which, in turn, rest on the main wall girders and longitudinal girders under the granite colonnade beneath the sidewalk. The column line beneath the granite colonnade has no floors attached to it except under the main entrance vestibule. To provide for the general stability of these columns, a horizontal stiffening system is introduced between the girders near the tops of the columns under the granite colonnade, and attached to the lower lengths of the columns under the main façade. A vertical bracing system is also introduced transversely between the girders and columns in these two lines, and is connected up to the columns which are provided outside of the latter under the entrance pavilions. This vertical bracing system is carried down to the column bases. A vertical bracing system is also introduced longitudinally between the columns of one short panel under the colonnade south of the north carriage driveway entrance, terminating in portal bracing above the under-clearance line. A similar panel of bracing is introduced under the colonnade just north of the south carriage driveway entrance, and this latter system is carried down to the column bases. Struts of four angles, double-latticed on all sides, are framed between all other steel columns under the granite colonnade, longitudinally, at approximately the elevation of the lower floor of the Seventh Avenue Building.

The lower lengths of the columns under the carriage driveways on 31st and 33d Streets are braced below the floors to the under-clearance line in tower formation. The columns of the carriage driveways are at intervals of 30 ft. 6½ in. from center to center longitudinally, 46 ft. from center to center transversely in the south driveway, and 50 ft. from center to center transversely in the north driveway. The floors of these driveways slope from the street entrances

FIG. 1.—STATION AREA, FROM EIGHTH TO NINTH AVENUE, SHOWING SUBSTRUCTURES.

FIG. 2.—CONCOURSE ROOF DURING ERECTION.

to the entrance to the baggage-room wagonways at a rate of more than 6%, and these inclined floors are framed in the following manner: Single-web girders, 72½ in. deep, are framed transversely between the columns, and are face-connected to the latter, with the tops of the girders about 12 in. below the top of the finished pavement. Two inclined longitudinal girders, with top flanges 2 ft. 4 in. below the top of the pavement, are framed between the webs of the transverse girders, thus dividing each panel transversely into three equal spans. The floor-beams are 15-in. I's, at about 6 ft. from center to center, framed across the carriage driveways, bearing on top of the inclined girders, and framed at the ends to the webs of the horizontal wall girders, which are framed longitudinally between the columns at suitable elevations to receive the floor-beams and the masonry base course, which change elevation in each panel to suit the inclined floor. The floor-beams are stayed at the intermediate girder bearings by bent plates set on the lower side of the incline, and riveted to the top flanges of the girders and to the webs of the floor-beams near the top of the latter. The tops of the floor-beams are set 13 in. below the top of the pavement.

The vaulted ceilings of the Seventh Avenue vestibule and arcade are constructed of angles suspended from the roof trusses. The ceiling angles at the intersection of the lunettes and the main longitudinal vault of the arcade ceiling have constantly changing planes in both directions, and the development of these taxed the ingenuity of the template maker.

The easterly train-sheds are supported by the wall columns of the arcade shops, the columns of the north wall of the south carriage driveway, and the columns of the south wall of the north carriage driveway. These train-sheds have pitched hipped roofs, and are each 152 ft. 8½ in. long and 115 ft. 6 in. wide. Each has a longitudinal monitor, 91 ft. 7½ in. long and 20 ft. 4 in. wide. The portions of these roofs adjacent to the arcade shops and driveways are depressed between the transverse trusses for a width of 17 ft. and made flat. There are flat connecting roofs with warped surfaces between the train-shed roofs and the walls of the restaurants and the Seventh Avenue Building.

The main transverse trusses of the easterly train-sheds are of the quadrangular type, 115 ft. long, with curved bottom chords and with

top chords laid at roof pitch, except across the monitors, where they are horizontal. These trusses are 30 ft. 6½ in. from center to center, are 14 ft. deep at the center, and have panel points 10 ft. 2 in. from center to center between the flat depressed roofs above referred to. The end panels adjacent to these depressed roofs have solid-plate webs. The end connections on each truss were shipped blank, and drilled in the field, using the column connection angles as templates. The trusses were shipped partly riveted up, with provision for two field splices in the top chords and three field splices in the bottom chords. The web members of the six central panels were shipped loose. The trusses were assembled at the site and completely bolted up, after which they were raised into position by the simultaneous operation of two independent hoists, one on either side of the roof structure. They were supported temporarily at their ends on seat angles riveted to the columns under the connection angles. The trusses were afterward tested for deflection, and correctly lined and leveled, after which the end connections were drilled and riveted. Rivets were substituted for bolts in the truss joints after the roof structure was completely fitted up and adjusted. Vertical sway frames, designed to act as purlins, were connected between the trusses at each panel point. A monitor frame was built up on top of the trusses and sway frames. Rafters, composed of two 6-in., 8-lb. channels, back to back, attached to the top chords of the purlins, divide each roof panel into three equal divisions. Between and flush with the rafters 6-in. channel jack-rafters are framed at intervals of about 5 ft. from center to center. The depressed portions of the roof are framed with 10-in. I-beams at intervals of about 6 ft. from center to center, supported on the bottom chords of the roof purlins on one end, and framed to the girders in the walls of adjoining buildings on the opposite end. This depressed portion of the roof is covered with vault lights; the remainder of the roof is covered with a glass skylight.

The general waiting-room is an immense cathedral clerestory, 320 ft. long and 112 ft. 10 in. wide, from center to center of wall columns, 143 ft. from the floor to the highest point of the ceiling, and 167 ft. from the floor to the highest point of the roof. The floor of this room is about 27 ft. above the top of rail, and a pipe gallery floor is framed beneath it and above the train clearance.

The general waiting-room is enclosed to an elevation about 37 ft.

FIG. 1.—STATION BUILDING STEEL DURING CONSTRUCTION.

FIG. 2.—EASTERLY TRAIN-SHEDS DURING CONSTRUCTION.

below the highest point of the roof by carriage driveways on the north and south, restaurants and an arcade vestibule on the east, and the sub-waiting-rooms and concourse vestibule on the west. In the four corners of the general waiting-room structure there are staircase towers, each 18 ft. 6 in. by 20 ft. 6 in. from center to center of tower columns. The ceiling is arched east and west between these towers, and is divided north and south between the towers into three immense vaults which have been constructed of bent steel beams suspended from channel-iron purlins framed between the trusses.

The roof is a series of intersecting gables, three transverse and one longitudinal. A simple mill-building type of frame was adopted for the lofty lantern of this room, with **A**-shaped main and gable trusses, 19 ft. and 21 ft. deep at the center, and knee-braced to the columns. Ridge trusses, 94 ft. long, and of the full depth of the gable trusses, are framed between the latter in each transverse gable. Longitudinal ridge trusses, each about 44 ft. long, and of the full depth of the main trusses and transverse ridge trusses, are framed between the latter. The valley trusses and groin trusses at the ceiling vault intersections are framed between the ridge trusses and the columns. All trusses have single web systems. The rafters are 10-in., 15-lb. channels, about 5 ft. 6 in. from center to center; bearing on and attached to the I-beam purlins which are framed between the gusset plates of the truss panel points.

The desired architectural effect of massive walls has been obtained by furring the interior face of the general waiting-room in steel framework, faced with stone and artificial stone, the masonry walls being placed on the outside. This method reduced the dead load to a minimum, and provided space for pipes, and also for heating and ventilating ducts.

The steel columns which support the vertical loads from the roof and ceiling are inside of the free-standing architectural columns and 9 ft. 5 in. from the center of the wall columns. The wall columns are directly back of these free-standing columns, and other wall columns are introduced opposite pilasters in the walls of the restaurants and sub-waiting-rooms, below the roofs of the latter, and in the mullions of the semicircular openings of the lantern walls. These mullion columns are in most instances offset from the lower wall columns, resulting in the introduction of heavy girders at the adjoining roof

levels to transfer the loads. Owing to the relation of the general waiting-room to the tracks, certain important wall columns are supported on girders at the general waiting-room floor level, and these girders, which are of full depth between the general waiting-room floor and the pipe gallery floor, transfer the loads to other columns founded at track level.

Wall beams and girders are introduced at the floor tiers, ceiling, and roof tiers of adjacent buildings, and otherwise at intervals of approximately 20 ft. in the height of the wall, for the support of the same.

The frame of the general waiting-room is designed to resist a horizontal wind pressure of 30 lb. per sq. ft. on the entire exposed surface of the lantern. A horizontal lateral system is framed between the main columns and the wall columns, at their tops on the east and west sides of the lantern, and connected up to the tower columns and to a lateral system of frames between the towers at the north and south ends, thus completely banding the top. The wall columns back of the main columns are connected to the latter above the adjoining roofs by a vertical system of bracing through which the stresses at the foot of the knee-braces on the main columns are transferred; and this vertical bracing system is connected at its foot through the adjoining roof trusses or directly to a horizontal system placed in the roofs of the sub-waiting-rooms on the west, and below the roofs of the restaurant on the east. These horizontal systems are in turn connected up to vertical bracing systems in the wall frames at the north and south ends of the sub-waiting-rooms and restaurants, and the effect of wind pressure on the east and west walls of the lantern is thus taken to the ground.

The tower columns are braced vertically and also transversely at every tier above the adjoining roofs, and the end columns of the towers and the intermediate columns of the north and south walls are connected to horizontal trusses built through the roof trusses of the attics over the carriage driveways. Two panels, each of vertical bracing and portal bracing, between the east and west wall columns, about midway of the wall lengths, north and south of the arcade and concourse vestibules, are provided to transfer to the ground the effect of wind pressure on the north and south ends of the lantern.

All wall columns of the lantern are designed to resist wind pressure

FIG. 1.—ARCADE ROOF STEEL DURING CONSTRUCTION.

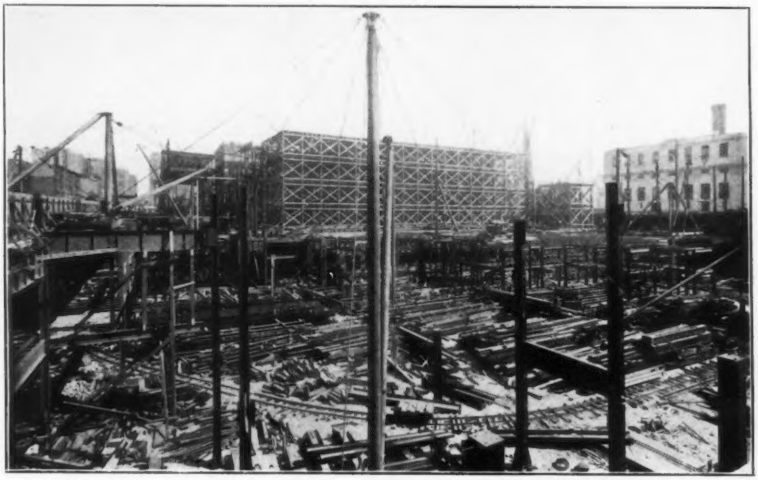


FIG. 2.—FALSEWORK FOR WAITING-ROOM ROOF, AND MASONRY DURING CONSTRUCTION.

in addition to vertical loads, and all are connected at the top and bottom, as described above, to the horizontal systems.

The effect of wind from any direction on the main frame of the lantern was determined and provided for.

The erectors chose to build a timber trestle from track level to roof level inside of the general waiting-room columns, the purpose being to use it first for the erection of the steel and masonry and afterward as a scaffolding from which to finish the ceiling and interior walls.

The erection of the lantern of the general waiting-room was not permitted until the surrounding sub-waiting-room, restaurant, and carriage driveway structures had been erected, plumbed, and fitted up, including the horizontal systems in or below their roofs, provided as above described, to band the foot of the lantern structure. The roof trusses were shipped knocked down, and were assembled in position by blocking above the trestle, a bolt being fitted into every hole in each joint. After the roof was completely fitted up, lined, leveled, measured, and correctly adjusted, rivets were substituted for bolts in all joints.

The concourse roof was a unique and unusual structural problem. It is made up of three barrels, the two outer of 85 ft. 10 in. span and the center barrel of 95 ft. 4 in. span, running east and west and intersected by a cross-barrel of 98 ft. span running north and south, thus forming, over the main concourse floor, a series of three groined vaults supported on steel columns. The concourse roof columns, east and west, are at intervals of 27 ft. 8 in. from center to center for the two bays west of the groined roof, and 26 ft. 8 in. from center to center for the single bay east of the groined roof.

The main roof structure abuts on the sub-waiting-rooms on the east, and is banded by a lower roof, constructed of a series of Guastavino and steel-framed vaults, on the other three sides, which, in turn, connect to the surrounding buildings. The architectural effect of these latter roofs, is enhanced by small steel arches framed between the columns below the Guastavino vaults.

The engineers' original design for the concourse roof provided for exposed ties in the outer barrels at the springing line, all parts to be riveted, but, owing to the desire of the architects to omit all exposed ties, the more expensive cantilever arch type, anchored to

the surrounding structure, was adopted for the arches, and single-web trusses with arch effect below the finished roof line, and with the remainder of the trusses extended above the roofs and forming monitors at their intersections, were adopted for the groins.

After the static determinations were completed, the effect of temperature was analyzed and provided for in all anchorage details.

The main columns are 3 ft. square, and some of these have an unsupported length of 68 ft. 9 in. They are made up of four 8 by 8-in. angles, double-latticed on all four sides, with double-latticed diaphragms, placed horizontally, at intervals of about every 10 ft. of the height of the columns, and provided with riveted bases which have vertical diaphragms. The tops of these unsupported lengths of columns have unique details for splicing the upper lengths, which latter are built with branches arranged to fit the arch trusses at the tops and designed to continue the intrados of each arch to the cap on the lower lengths of the columns. The column splice detail referred to was arranged so that the upper length of columns could be slipped on over it and take bearing on the cap of the lower length. The rivets in these splices were driven by the aid of hand-holes in the upper lengths of the columns, and neatly-made plates were tapped on to the columns flush with their faces after the riveting was completed.

Each half of the main cantilever arch trusses was shipped in two sections, with field splices in both top and bottom chords located in the panel on the column side of the anchorage tie connection. At the crown joints ½ in. play was left between the two complete halves of each arch truss, and fillers were provided for adjustment between web connection angles. The main arch trusses throughout the structure have double web systems, with solid plates in the panels near the crown, and double intersecting angle diagonals and double angle radial members in the remaining web panels. The bottom chords are double-laced between the panel points by angle lacing, with angle struts transversely at each panel point. The top chords are covered with riveted plates. The web systems of these trusses are about 2 ft. from center to center; the trusses are about 2 ft. deep, from back to back of angles at the crown, and have a depth of about 6 ft. 7 in., from back to back of angles measured radially at the highest point of the column attachment. Horizontal ties, with vertical

FIG. 1.—CONCOURSE ROOF DURING CONSTRUCTION.

FIG. 2.—CONCOURSE ROOF DURING CONSTRUCTION.

strut supports above the columns, are framed between the center and outside cantilever arches of the three east and west barrels, with attachment to the top chord of the arch trusses at the quarter points. Triangular frames, with similar attachment to the arch trusses, also attached to the columns in the north and south building walls, and strutted back against the concourse columns above the Guastavino dome roofs, form the terminal anchorages of the outside arches of the east and west barrels. Between the columns which support the three east and west barrels, small double-latticed ornamental arches with two webs are framed just above the lower lengths of the columns. Above these arches a system of double-latticed struts, with two webs and ornamental portals, hold the upper lengths of the columns rigidly together. The main cantilever arches of the north and south intersecting barrel were made in two halves, and each half was shipped in two pieces, with field splices in the third panel above the columns. The anchorage ties for these arches are attached much nearer the top than in the case of the other arches hereinbefore described. The stresses from these ties are transmitted through the frames at the tops of the opposite columns, and thence through the triangular terminal anchorage frames to the columns in the Eighth Avenue Building, and in the west face of the concourse roof, on the one hand, and through the medium of frames built between the sub-waiting-room roof trusses into the columns on the east face of the concourse, and in the west wall of the general waiting-room, on the other hand.

The intermediate columns, placed in the mullions of the tympana at the vertical faces of the main concourse roof, have slotted connections at their tops to the bottom chords of the cantilever arches.

The diagonal trusses at the groined intersections are about 17 ft. deep at the apex joints and about 13 ft. 7 in. deep, measured radially, at the knuckles in the top chords. These trusses have redundant members in order to give the desired arch effect below the roof line. They were shipped knocked down, and were assembed at the site, a bolt being fitted in every field hole, after which they were raised into position. One diagonal truss in each panel was assembled complete and designed to support the two halves of the opposite diagonal truss which were framed to it at the apex joint. Between the main truss panel points 12-in. I-beam purlins are framed, and are banded by two equally-spaced steel plate and angle trough gutter

ribs riveted to the top flanges of the I-beams in each bay. The skylight frames of bent T's are mounted on, and riveted to, these gutter ribs.

All arches were completely assembled and fitted at the shop before shipment. Each half of the diagonal trusses was similarly assembled and fitted at the shop. All parts of the roof were joined together in the field without difficulty, except at the apex joints between the diagonal trusses, where it was necessary to substitute some new gusset-plates and drill the connections in the field. This was undoubtedly due to slight inaccuracies in shop work, which might not have been apparent in a less complicated roof.

At the main concourse floor level, a timber platform was erected, and on it three tower travelers were mounted. The roof was erected with these travelers. Tower props, each about 12 ft. square, were built up from the timber platform for the support of the apex joints of the diagonal trusses pending their adjustment.

The erection of the concourse roof was not permitted until the surrounding structures to which it was anchored had been completely erected. The terminal anchorages, with the corresponding halves of the cantilever arches in the two outer barrels of the three-barrel formation, were first erected, and were held by derrick hitches from the travelers until the intermediate column and cantilever arches were placed. The groined intersecting barrel was erected after the remainder of the roof frame had been set up. After the columns had been plumbed and the roof structure lined, leveled, measured, and completely fitted up, all joints were riveted, the first portions completed being the three east and west barrels; the final closure was made in the north and south intersecting barrel.

The concourse roof and all other portions of the Station Building were riveted up without expansion joints. The effect of temperature during the erection of the steelwork of the Station Building was provided for by selecting certain definite lines of adjustment, namely: on the north side of the columns in the north wall of the 31st Street Building, on the south side of the columns in the south wall of the 33d Street Building, on the east side of the columns on the east side of the Eighth Avenue Building, on the west side of the columns in the west wall of the general waiting-room, and on the east side of the columns in the east wall of the general waiting-room.

In these cases the connecting pieces were either shipped blank and drilled in place, or connection angles were made from field measurements, according to the type of connection. Fillers were also provided where required. Riveting of the portion of the concourse floor connecting to the main roof columns was not permitted until after the roof had been riveted up.

All columns were set accurately and all work was laid out with true relation to the governing axes of the principal architectural features. Each feature of the structure was completely fitted up, plumbed, and measured before riveting was permitted.

The many heavy columns developed riveted base details of a very complicated character, oftentimes difficult to manufacture, particularly as regards the fit of the base stiffener angles. In some instances, the removal of these for lack of fit would have required rebuilding entire column lengths. The manfacturers, therefore, were permitted to substitute slab bases, planed on top and bottom, with riveted wing plates and base angles, producing a very satisfactory detail. The slabs vary from 2 to 6 in. in thickness. A series of tests was made at the mill to determine the permissible stress for the slabs, and 15 000 lb. per sq. in. was adopted, the tests showing an elastic limit and ultimate strength equal to the manufacturers' standard material.

The manufacturers divided the structure into twenty subdivisions, numbered from *C*-2 000 to *C*-2 020, inclusive, and all shop drawings were identified by the mark of the subdivision which they illustrated; the material was similarly identified during manufacture, shipment, and erection.

In addition to designing the steelwork and checking the shop drawings, the engineers maintained general oversight of the manufacture, kept records of the progress of the same, estimated the weight of every piece of material for comparison with the scale weight, kept records of shipments and deliveries, and supervised the erection.

Extracts from the specifications for the manufacture of the Station Building steel are given in Appendix C.

PLATFORMS.

All platforms are of reinforced concrete slabs, varying in thickness at the center from 8 to 16 in., spanning between the 16-in. concrete side-walls, and overhanging the latter 1 ft. 8 in. at each edge, which

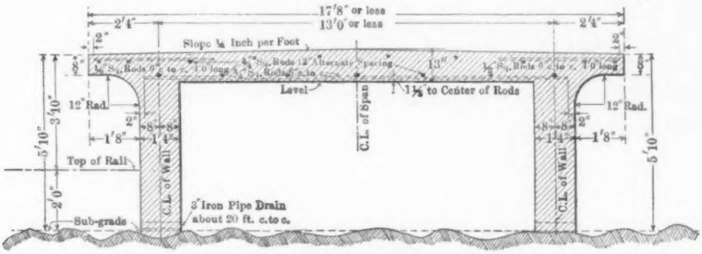
is 8 in. thick. The overhanging edges have coved soffits of 12 in. radius.

Two intermediate bearing walls of concrete, each 16 in. thick, divide the reinforced slab of the wide platform No. 2 into three equal spans.

The top surfaces of the platforms have granolithic finish, jointed in squares of about 30 in. on a side, and stippled to avoid slippery tread.

Exposed wall surfaces were rubbed after the forms were removed.

The platforms were designed to support a live load of 300 lb. per sq. ft. The slabs are reinforced transversely with steel equal in area to 0.007 of the area of the concrete, and also reinforced longitudinally with the same percentage of steel to resist temperature stresses.



TYPICAL SECTION OF REINFORCED CONCRETE PLATFORM
(13″SLAB)
SCALE IN FEET
0 1 2 3 4 5

FIG. 6.

The reinforcement is of plain square rods, 6 in. from center to center transversely and ¾ in. above the bottoms of the slabs, with an overlapping set of rods extending into the overhangs near the tops of the slabs.

The longitudinal rods are staggered, one set being placed just over the bottom set of transverse rods and the other near the tops of the slabs and about 12 in. from center to center. The supporting walls are of plain concrete. The platform concrete is composed of 1 part Portland cement, 2 parts sand, and 4 parts broken stone or clean gravel which would pass through a ¾-in. ring. The granolithic top dressing is ¾ in. thick, and is made of 1 part Portland cement and 1½ parts sand, hand-floated, trowelled smooth, and having a stippled finish.

The platforms have numerous openings for elevators, and are fitted with hatches over the elevator pressure tanks. These openings and

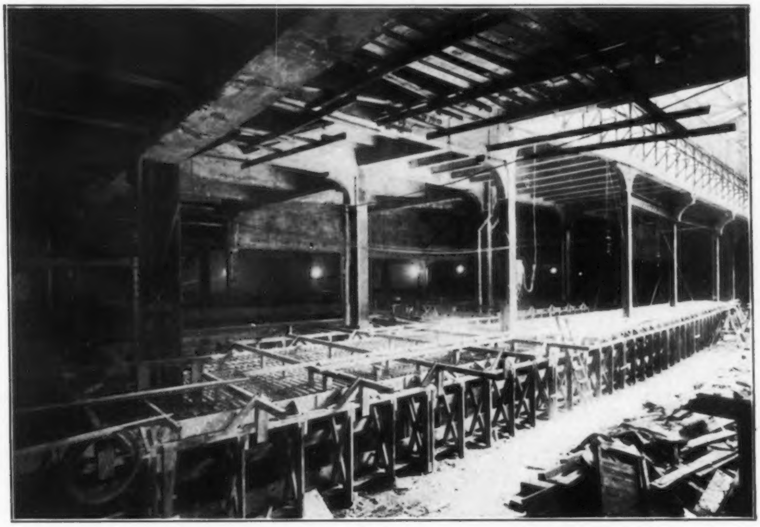


FIG. 1.—PLATFORMS DURING CONSTRUCTION.

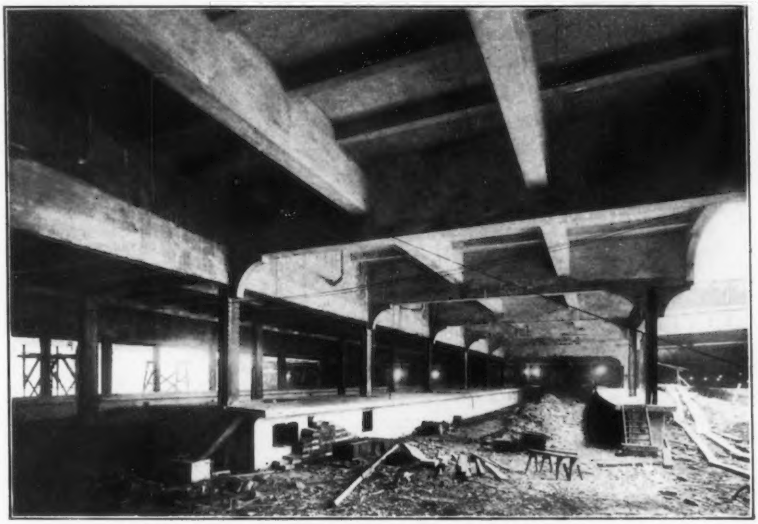


FIG. 2.—PLATFORMS DURING CONSTRUCTION.

hatches are framed in steel. The hatch covers are of thin reinforced concrete slabs finished like the remainder of the platform surfaces.

Openings for the installation of signal apparatus, openings for ventilation, fitted with wire screens in cast-iron frames, and openings for access, each fitted with two wrought-iron swing doors hung on rebated cast-iron frames, were built into the side-walls of the platforms.

Bronze sockets with hinged covers were built into the platforms along curved edges at intervals of 4 ft. to receive posts of pipe railing in case the latter is required when the platforms are in service. Bent wrought-iron pipe anchors, threaded for $\frac{7}{8}$-in. bolts, were built into the overhanging edges of the platforms on curves at intervals of 2 ft. from center to center to provide for the attachment of a wooden edge strip, if it is required.

The spaces under the platforms are used as pipe galleries for longitudinal service lines distributed between the pipe subways; and pipe sleeves were built into the platform walls for service taps to the adjacent tracks.

There are weep-holes through the platform walls at sub-grade, for drainage. The platforms were built in monolithic sections, each about 30 ft. long. Their total area is 244 270 sq. ft.

Base Lines and Levels.

It may prove interesting to observe that New York City blocks are not exact parallelograms, although the errors are so small that the irregularity is only observable on the records.

The center line of the terminal east and west, while apparently coincident with the center line of 32d Street, is not exactly so, but rather an average adjustment of the direction of the center lines of 31st and 33d Streets, both opposite the terminal and for blocks east of Seventh Avenue and west of Tenth Avenue. This main base line was determined on the ground, and permanently monumented. Parallel base lines, east and west, as well as transverse base lines, north and south, 100 ft. apart, were established, and all working drawings were related thereto.

A profile grade line for the top of rail was established in connection with the necessary tunnel grades east and west of the terminal. An understanding was reached with the engineers of the city, which fixed the surrounding curb grades in the streets, and these were made

to conform with existing street construction reduced to practicable mathematical lines.

Bench-marks were established from time to time, as found necessary. The datum plane conforms with that of all the other construction work of the company, + 300 being practically mean high water in the harbor.

Engineering Organization.

George B. Francis, M. Am. Soc. C. E., acted as the engineering executive for Westinghouse, Church, Kerr, and Company, with Joseph H. O'Brien, M. Am. Soc. C. E., as Resident Engineer directly in charge of the design and with engineering supervision of the construction.

Westinghouse, Church, Kerr, and Company reported directly on all work, excepting the steel for the Station Building, to George Gibbs, M. Am. Soc. C. E., Chief Engineer of Electric Traction and Station Construction of the Pennsylvania Tunnel and Terminal Railroad Company.

A force of engineers and draftsmen at headquarters designed the work herein described and another corps was maintained in the field for engineering service in execution. The following persons were connected with the engineering department, and performed faithfully the duties entrusted to them:

Mr. F. F. Skinner, D. Y. Dimon, M. Am. Soc. C. E., H. S. Devlin, Assoc. M. Am. Soc. C. E., W. A. Nial, Assoc. M. Am. Soc. C. E., and C. E. Hunt, Assoc. Am. Soc. C. E.

William R. Webster, M. Am. Soc. C. E., was employed to care for the mill and shop inspection of the steelwork.

All inspectors on the various items of work were under the supervision of the engineers.

The designing of the steel for the Station Building was done by Westinghouse, Church, Kerr, and Company, under an engineering engagement with McKim, Mead, and White, the Architects.

APPENDIX A.

EASTERLY PORTION.

Cement.—All cement used in the work shall be Portland cement furnished by the Railroad Company in accordance with conditions of Article 12 of the contract.

Sand.—Sand for mortar or concrete shall be coarse, sharp and silicious, not containing more than 1.5 per cent. of mica, loam, dirt or clay, or of all combined, and equal in quality to the best Cow Bay sand. If required by the Engineers, it shall be screened.

Broken Stone.—Sound rock excavated from the work may be used if hard, crystalline, and practically free from mica. If a sufficient amount cannot be obtained from the work, sound trap or hard limestone must be furnished. Gneiss from other localities will not be accepted. The rock must be clean when delivered at the crusher. It shall be broken by machine and screened in a rotary screen, which will remove all dust and fragments, which will pass through a hole $\frac{3}{8}$ inch in diameter, and all pieces which will not pass through a hole one and one-half ($1\frac{1}{2}$) inches in diameter; all fragments between these limits will be retained. In special cases, hereinafter specified, the maximum size will be that which will pass through a hole $\frac{3}{4}$ inch in diameter.

Mortar and Grout.—In proportioning materials for mortar, grout and concrete, one volume of cement shall be taken to mean three hundred and eighty (380) pounds net, one volume of sand or broken stone shall be taken to mean three and one-half ($3\frac{1}{2}$) cubic feet packed or shaken down. Sand and broken stone shall be measured in barrels or rectangular boxes. Measurements in wheelbarrows will not be permitted.

In preparing mortar for brick or stone masonry, the specified amounts of cement and sand shall first be mixed dry to a uniform color. The water shall then be added in such a manner as not to wash out any of the cement and the mixing proceeded with until the mortar is thoroughly mixed and of uniform consistency.

Grout will generally be in the proportion of one (1) part of cement to one (1) part of sand by volume. The materials shall be thoroughly mixed dry and water then added while the mixing proceeds, until the grout is of the required consistency. The mixing shall be continued vigorously, preventing the separation of sand, until the entire amount mixed is used. Grout shall be used about the imbedded steel for filling small voids wherever concrete cannot be properly used and wherever required by the Engineers to insure tight work.

Masonry—Concrete.—Concrete for abutments and masonry tunnels shall be in the proportion of one (1) part of cement to two (2) parts of sand and four (4) parts of broken stone by volume. All other concrete, except that required for grillages, concrete steel floor plates, and

base of granolithic, shall be in the proportion of one (1) part of cement to two and one-half ($2\frac{1}{2}$) parts of sand and five (5) parts of broken stone by volume. The concrete for grillages and concrete steel floor plates shall be in the proportion of one (1) part of cement to two (2) parts of sand and four (4) parts of broken stone that will pass through a hole $\frac{3}{4}$ inch in diameter.

Whenever practicable, concrete shall be machine mixed. The mixing machine shall be a rotary mixer and of a pattern that will mix the concrete in batches and permit the definite measurement of the materials for each batch. When the Engineers consider it impracticable to mix by a machine, it may be mixed by hand as follows: The mixing shall be done on a platform of boards or planks securely fastened together. The mortar shall first be made as hereinbefore specified. The broken stone previously wetted shall then be added and the mortar and stone turned over with shovels until the mortar is uniformly distributed through the mass and every stone is coated with mortar.

The degree of moisture for mortar, grout and concrete shall be at all times as required by the Engineers or their Inspector; in general, mortar shall be plastic, grout shall be fluid enough to be pumped, and concrete shall be of such consistency that it will quake when being deposited, but not wet enough to cause the mortar to separate from the mixture.

Concrete shall be deposited in the work in such a manner as not to cause separation of mortar and stone. It shall be laid quickly in layers not exceeding nine (9) inches in thickness and thoroughly rammed with rammers of such form and material as the Engineers may approve; special shaped rammers will be required for corners and other places where ordinary rammers would not be effective. Compact, dense concrete must be obtained with all the voids between the stones filled with mortar. If voids are discovered at any time, the defective concrete shall be removed and immediately replaced by concrete of such mixture and in such manner as the Engineers may direct.

Contraction joints in concrete formation shall be made where the Engineers may require them. Where columns limit the thickness of "concrete" at face, the cement mortar used in conjunction with metal reinforcement is measured and paid for as concrete only.

When the placing of concrete is suspended, the Engineers may require a joint to be formed in a manner satisfactory to them, so that the fresh concrete when added, may have a bond. Before depositing fresh concrete, the entire surface on which it is to be laid shall be cleaned, washed, brushed and slushed over with grout of cement without sand.

The surface of freshly laid concrete shall be protected from injury in such manner and for such time as the Engineers may require; concrete injured in any manner shall be removed.

Water used in mortar, grout and concrete shall be clean fresh water.

No mortar, grout or concrete which has commenced to set shall be used anywhere in the work. Re-tempering of mortar or grout which has commenced to set will not be permitted.

Forms for concrete shall be substantial and must preserve their accurate shape until the concrete has set. Where the concrete will show in the finished work, the face of the form shall be built of matched and dressed planking finished truly to the lines and surfaces shown on the plans. Adequate measures will be taken to prevent the adhesion of mortar to the forms. Forms which have become warped or distorted shall be immediately replaced. All forms of whatsoever nature required for concrete work shall be furnished, erected and removed by the Contractor whose price for concrete per yard must include the cost of the forms required.

Contractor shall not remove forms for concrete work until the concrete has set up to the satisfaction of the Engineers. Any forms left in the work by direction of the Engineers shall be paid for by the thousand (1,000) feet, board measure, of the completed form measured in the work.

Faces which will show in the finished work shall be true to the form intended and shall be wholly smooth, free of ridges and cavities, due to shortage of mortar at the face.

Exposed faces shall have a facing of mortar two (2) inches thick deposited simultaneously with the corresponding layers of concrete and separated from the concrete by a metal diaphragm of approved form. After the mortar and concrete have been deposited, the diaphragm shall be removed and the materials well worked together by spading and tamping, so as to insure their bonding. In places where this method cannot be used, as the under surface of arches, the same end shall be attained by methods satisfactory to the Engineers. Plastering the face after removing the forms will not be permitted unless otherwise provided. The facing mortar shall be of the same composition as the mortar used in the concrete back of same.

Rock surfaces shall be thoroughly washed and cleaned before concrete is deposited against them. Earth surfaces shall be wetted and compacted by ramming immediately before depositing concrete thereon.

If leaks appear on the surface of the concrete at any time after removing the forms, the Contractor shall remove the concrete through which the water passes and replace it with sound concrete and shall stop the leakage or conduct the water to the floor of the tunnels through channels or pipes in the concrete, or take such other measures as the Engineers may require.

The rough concrete under grillage seats shall be brought up to within two and one-half ($2\frac{1}{2}$) inches of the underside of the grillage bearing plates and shall then be top dressed with Granolithic two (2)

inches thick, leaving a joint to be made not over ½ inch thick between the top surface of the Granolithic and the underside of the grillages.

Granolithic shall be laid on a solid concrete base, and shall have two (2) inches total thickness. The base of this Granolithic shall be one and one-half (1½) inches thick, made up in proportions of one (1) part of cement, two (2) parts of sand, and four (4) parts of broken stone, the maximum size of which will pass through a hole ¾ inch in diameter. This base shall be carefully laid and screeded in place to insure a level top surface. Upon the base so prepared, the Contractor will lay a top dressing of grout ½ inch thick. This grout shall be hand floated and troweled smooth to give a durable and true top surface.

Terra Cotta Block Walls.—The terra cotta blocks used for the backing of waterproofing behind the abutments, shall be hollow, porous, terra cotta partition blocks. These blocks must be so burned that their walls will be absolutely porous.

Samples of the terra cotta blocks to be used in the work must be submitted to the Engineers before they are purchased by the Contractor, and all of the blocks used in the work must be sound, of true mold and in conformity with the sample selected by the Engineers.

Terra cotta blocks shall be laid as far as possible on their smaller beds so that the hollow space may be continuous vertically. Joints shall be broken every fourth course as shown on the drawings. The blocks shall be laid in mortar made of one (1) part Portland Cement, one (1) part best Rockland lime and three (3) parts sand. The bed joints shall be buttered and the work shall be laid up with such care that mortar will not accumulate in the holes of the blocks. The Contractor shall keep the hollow spaces clean to the satisfaction of the Engineers. Joints shall not be over ⅜″ thick and shall be struck with a trowel.

Granite and Granite Masonry.—All pier caps, templates and pedestals, where so shown and required by drawings, shall be of cut granite.

Granite must be strong, sound, compact, moderately fine grained, of uniform quality and appearance, and free from any defect which in the judgment of the Engineers may impair its strength and durability.

The bottom bed shall always be of the full size of the stone and no stone shall have an overhanging top bed.

The stones shall be cut to the dimensions shown on drawings.

The face lines of each stone shall be true, and the rise as fixed by the face line shall not vary anywhere more than $\frac{1}{16}$ of an inch from the true rise shown on the drawings.

The upper and lower beds shall be truly parallel. The lower bed shall have no depression within six inches of the face and none exceeding one inch in depth nor more than six inches in its greatest dimension over the remainder of the bed. Depressions of more than

$\frac{1}{4}$ in. below the true plane shall never exceed over $\frac{1}{8}$ of the area for the bed.

Vertical joints shall be cut to exact lines on the upper bed face and for 6 inches back from the face of the lower bed. The remainder of the joint shall be cut to conform to the requirements for the lower bed.

The upper bed face and lower bed (of pedestals under girders) for a distance of six inches back of the face shall be bush-hammered, six-cut with true lines and surfaces.

The upper bed and the lower bed of pier caps and templates for a distance of six inches back from the faces shall be bush-hammered, six-cut with true lines and surfaces.

These pier caps and templates shall have rock faces. No grab hole or lewis hole shall be made in the bed or face of any stone.

Stones shall be set in full beds of mortar containing 1 volume of cement to $2\frac{1}{2}$ volumes of sand. All stones must be carefully cleaned and wet before setting.

No stone shall be laid in freezing weather, except with the express permission of the Engineers, and under such conditions as they may impose to secure immunity from injury by freezing.

Joints shall be caulked temporarily with oakum or rope yarn and filled with a thin mortar of the same composition as used for beds well worked in with a sword so as to completely fill the joints.

All the joints shall be cleaned out to a depth of $1\frac{1}{2}$ inches and pointed in mild weather with a mortar containing one volume of cement to two parts of sand, mixed with a small proportion of water and driven into the joint with a calking iron.

All granite masonry foundation walls shall be built of dimension stones laid on a full and sufficient bed of Portland cement mortar made in proportions of one volume of cement to $2\frac{1}{2}$ volumes of sand.

All joints to be thoroughly filled with cement and carefully chinked with slate spalls.

All stones to be derrick laid and no joints to exceed two (2) inches in thickness.

All joints shall be thoroughly broken as shown on the drawings.

The measurements on all granite and granite masonry will be taken in cubic contents to the draft lines and no allowance made for rock face.

Vitrified Conduits for Electric Cables.—The vitrified conduits for electric cables shall be manufactured of the best clay, thoroughly burned, sound in all respects, straight and free from splits, fractures, soft spots, stones, cracks or blisters tending to impair their strength or durability. They shall be thoroughly and completely glazed with good salt glaze. The interior surfaces of the duct holes shall be smooth and free from any projections or imperfections which may tend to strip the lead coating from the electric cable when pulled through the duct. The ends shall be cut smooth and square with the axis. Ends of holes

shall be slightly bell-mouthed. When conduits are cut to special lengths, the cut end must be dressed with chisel and rasp until the hole is slightly bell-mouthed and has smooth edges.

Conduits shall be of whatever form and pattern the Engineers may require, either single or multiple duct conduits. In the four-way conduits, dowel holes are to be formed at each end for truly centering the sections when laying. The standard length of single-duct conduits shall be eighteen (18) inches and four-way conduits thirty (30) or thirty-six (36) inches, as the Engineers may determine from the samples submitted. The lengths must not vary from the standard by more than $\frac{3}{4}$ inch. The duct holes in single-duct conduits shall not be less than three and one-half ($3\frac{1}{2}$) inches nor more than three and seven-eighths ($3\frac{7}{8}$) inches in diameter, or square with corners rounded; the outside walls and webs shall be $\frac{3}{4}$ inch thick. The outside dimensions of four-way ducts shall not exceed nine and one-half inches on a side. The conduits shall be square on outer lines with corners rounded.

The adopted sample sections of conduits exhibited at the office of the Engineers shall represent in every way the conduits to be furnished by the Contractor.

The conduits shall be laid in about $\frac{1}{4}$ inch beds of mortar and in perfect alignment and grade throughout. The vertical joints between conduits shall be filled with mortar and the concrete carried up in layers as the conduits are laid. Dowels with central washer shall be provided by the Contractor and placed in every dowel hole. A wooden mandrel four (4) feet long, the center dimensions of which are $\frac{1}{8}$ inch less than the bore of the conduits, the end dimensions of which are $\frac{1}{2}$ inch less than the bore of the conduits, shall be threaded through each hole after the conduits have been bedded in place, and the Engineers may require the Contractor to thread the mandrel through a second time. A spring steel tube scraper with a flue brush behind it, or other device approved by the Engineers, shall be attached to the end of each mandrel and used to remove all foreign matter from the ducts. All ducts must be thoroughly cleaned to the satisfaction of the Engineers.

Butt joints of conduits shall be broken at every tier half the length of the section, or as may be specially required by the Engineers. Every joint of the four-way conduits shall be lapped around with two (2) thicknesses and six (6) inches overlap of No. 6 cotton duck canvas six (6) inches wide, three (3) inches on each abutting section, saturated with neat cement grout immediately before placing. These laps are to be doubled on curves. Single-duct conduits are to be lapped on curves only.

Short sections of conduits shall be used at manholes only to effect proper bond and closures. These short sections shall be cut cleanly and truly square across without splitting the ducts. No four-way conduit section shall be cut less than twelve (12) inches long, and no single conduit section shall be cut less than nine (9) inches long.

Paraffined wooden plugs shall be furnished by the Contractor and placed in the free ends of all ducts when work is discontinued and shall be left in place.

Electric conduits shall be measured in the work and the unit price shall include dowels, canvas lapping and paraffined plugs.

Manholes will be built at intervals and in accordance with plans furnished by the Engineers. Iron manhole castings and covers, doors and other details as shown on the plans shall be supplied and set in place by the Contractor.

One No. 8 B. and S. gauge galvanized iron wire shall be placed, pulled and left in each duct from manhole to manhole.

Conduits will be enveloped in waterproofing laid around the encasing concrete where shown on drawings, and in the manner hereinafter specified under "Waterproofing."

Wrought Iron Pipe for Electric Ducts.—Where iron pipes are required for electric ducts, they shall be standard wrought iron, lap-welded pipes, three and one-half ($3\frac{1}{2}$) inches diameter inside.

Bent pipes shall be free from distortion in cross-section, and the bends shall not vary anywhere more than one (1) inch from the form required.

The ends of pipes shall be smoothed and rounded with a file on the inner edges, so as not to injure the lead covering of the electric cables, when drawn through. Paraffined wooden plugs shall be furnished by the Contractor and placed in the free end of all ducts when work is discontinued and shall be left in place.

Waterproofing.—It is intended that the interior of water-proof structures shall be permanently free from moisture or discoloration due to the percolation of water or other liquids from outside sources. This end shall be attained by means of a continuous flexible water-proof sheet surrounding the exterior of the structures (as shown by drawings).

Pitch used shall be straight run coal-tar pitch, which shall soften at 60 degrees F. and melt at 100 degrees F.; being a grade in which distillate oils distilled therefrom shall have a specific gravity of 1.105.

The felt shall be "Hydrex" felt manufactured by F. W. Bird & Son, East Walpole, Mass., or felt equally satisfactory to the Engineers.

Pitch, when applied, shall be of a temperature of not less than two hundred fifty (250) degrees F. The pitch shall be mopped on the surface of the masonry to a uniform thickness of not less than $\frac{1}{16}$ inch. Each layer of pitch must completely cover the surface on which it is spread without cracks or blowholes. The felt must be rolled out into the pitch while the latter is still hot and pressed against it so as to insure its being completely stuck to the pitch over its entire surface. Great care must be taken that all joints in the felt are well broken, and that the ends of the rolls of the bottom layer are carried up on the inside of the layers on the sides, and those of the roof down on the out-

side of the layers on the sides, so as to secure the full laps herein specified.

Waterproofing must be protected against injury at all times to the satisfaction of the Engineers.

Any waterproofed structure that is found to leak at any time prior to the completion of this contract shall be made tight by the Contractor in a manner satisfactory to the Engineers.

Waterproofing shall consist of six (6) layers of felt and seven (7) layers of pitch alternating, each strip of felt to lap not less than one (1) foot upon the previously laid strip and each section of waterproof sheet shall lap at least one (1) foot with the adjoining section.

Waterproofing will be measured by the square of one hundred (100) superficial feet and paid for accordingly.

Erection of Steel.—All steel will be furnished by the Railroad Company, delivered at a North River dock situated between West 29th and West 39th Streets, inclusive, Borough of Manhattan, City of New York. The Contractor shall receive the steel so delivered and transport it to the site of the work.

Contractor shall erect all the structural steel shown on or required by drawings and paint the same the specified number of coats of paint, and shall set all anchor bolts and do all field riveting required. Before, or during erection, all material shall be laid on skids above the ground so as to be kept clean. It shall be handled so as to avoid injury to the material. Any piece showing the effects of rough handling shall be repaired or replaced by the Contractor at his own expense.

Contractor shall furnish all equipment necessary and shall furnish and build all false works required for the proper erection of the steel work.

All grillages shall be set, filled and encased in Portland Cement concrete and grout (prior to the erection of the superimposed steel work) to the exact lines and grades given by the Engineers. Grillages so set shall be maintained in true position by the Contractor, and no steel work shall be placed upon the same until the Engineers so direct.

The floor plate which rests on the plate girder bridging shall consist of rolled steel beams imbedded in concrete as shown on the drawings. The bottom layer of concrete shall be laid in place by the Contractor before the floor beams are erected. The beams may be of varying lengths, not less than twelve (12) nor more than thirty (30) feet long, and they shall be spaced as shown on the drawings, breaking joints every six (6) or eight (8) beams. All of these beams must be straight and clean when laid in place.

Rivets shall be $\frac{7}{8}$ of one inch in diameter unless otherwise shown on the drawings. They shall have full hemispherical heads and shall completely fill the holes and be true and perfect in every way. Any loose, burned or otherwise defective rivets discovered at any stage of the

work shall be cut out and replaced by the Contractor. Field rivets shall be driven by pneumatic riveters wherever possible.

Slight misfits incidental to all erection, requiring reaming of unfair holes, etc., shall not be considered extra work.

All of the steel work to be incased in concrete except columns, and except as hereinafter specified, shall not be painted. The Contractor shall remove all dirt or filth which may be found on the steel delivered to him prior to erecting the same, to the satisfaction of the Engineers. He shall also clean all steel work which may be otherwise damaged after delivery to him. After erection, all steel work except grillages, floor plate, and those portions of plate girders imbedded in concrete, shall receive three (3) good coats of graphite paint of colors to be selected by the Engineers, or one (1) coat of "Tockolith" and two coats of No. 49 R. I. W. paint; each coat to be allowed to dry before another coat is applied. These three (3) final coats shall be applied immediately before erecting the girders, to surfaces which will not be accessible after erection.

Graphite paint to be used on the work shall be Dixon's Silica Graphite Paint, manufactured by Joseph Dixon Crucible Company, Jersey City, N. J., or other graphite paint of equal quality and satisfactory to the Engineers.

"Tockolith" to be used on the work shall be the Portland Cement paint of that name manufactured by Toch Brothers of New York City.

R. I. W. paint shall be the paint of that name manufactured by Toch Brothers, New York City.

The vehicle of the above graphite paint shall be boiled linseed oil. No adulterated or thinner oils shall be used. The boiled linseed oil must be absolutely pure, containing no matters volatile at two hundred (200) degrees F. in a current of hydrogen; shall not contain any resin or manganese and shall be perfectly clear upon delivery, and on standing no deposit should form, providing the oil be kept at a temperature of forty-five (45) degrees F. The film left after flowing the oil over glass and allowing it to drain in a vertical position must be dry to the touch after twenty-four (24) hours.

No painting shall be done in wet or freezing weather and no paint shall be applied to damp surfaces, or surfaces which are not thoroughly clean or dry.

APPENDIX B.

STREET BRIDGING STEEL.

Proportion of Parts.—No material shall be used less than $\frac{3}{8}$ of an inch thick, except that lattice bars or sway bracing may be $\frac{5}{16}$ of an inch thick, and except as may be required for lining and filling.

The various parts of the structures (except the 9th Avenue viaduct) are proportioned to sustain the following unit strains given in pounds per square inch.

Tension = 17,000 lbs. (net section).

For determining the net sections, rivet holes are assumed to be of $\frac{1}{8}$ inch greater diameter than the cold rivet.

$$\text{Compression} = \frac{17{,}000 \text{ lbs.}}{1 + \dfrac{L^2}{11{,}000\, r^2}}$$

Wherein L is the length of the member in inches and r is the least radius of gyration in inches.

No combination of stresses shall exceed 17,000 lbs. per square inch.

Shear in webs of plate girders (gross section)..10,000 lbs.
Rivet bearing ....22,000 "
Shearing strain on rivets shall not exceed......11,000 "

For field connections the number of rivets thus found shall be increased 25 per cent. if driven by hand, but increased 10 per cent. if driven by power.

The various parts of the 9th Avenue viaduct are proportioned to sustain the above unit strains for all loads excepting the reactions at the base of the Elevated Railroad pillars for the support of which the parts of the viaduct affected thereby are proportioned to sustain the following unit strains given in pounds per square inch:

Tension = 9,000 lbs. (net section).

For determining the net sections, rivet holes are assumed to be of $\frac{1}{8}$ inch greater diameter than the cold rivet.

Compression = 9,000 lbs.

Reduced by Gordon's formula using a factor of safety 5.

Sheer in Webs of Plate Girders, 7,500 lbs.

Rivet Bearing = 15,000 lbs.

Shearing Strain on Rivets = 7,500 lbs.

For field connections the number of rivets thus found shall be increased 25%, if driven by hand, but increased 10%, if driven by power.

All girders shall have a sufficient number of end stiffeners to transmit vertical shear into the web, and all rivets in these end stiffen-

ers shall be counted upon to take this vertical shear. Intermediate stiffeners shall be used at points of superimposed concentrated loading. Rivet pitch for stiffeners shall not exceed $4\frac{1}{2}$ inches. Stiffeners shall be used on all girders, the webs of which are less in thickness than $\frac{1}{60}$ of their unsupported depth, also on all girders the webs of which sustain a greater load than is allowed by the following formula:

$$P = \frac{11{,}000}{1 + \dfrac{d^2}{3{,}000\ t^2}}$$

In which

P equals allowed strain per square inch of section,
d equals unsupported depth of web in inches,
t equals thickness of web in inches.

Intermediate stiffeners shall be spaced not more than 5 feet apart, usually at intervals equal to the depth of the girder, and as shown on detail drawings.

Girders which carry masonry arches shall be provided with shelf angles as required by drawings.

In calculating shearing strains and bearing strains on web rivets of plate girders the whole of the shear acting on the side of the panel next to the support is considered to be transferred into the flange angles in a distance equal to the depth of the girder.

Where cover plates are used, one cover plate must be of full length of girder on top flange where shown or noted.

Girder bearing plates are so proportioned that the pressure upon the bridge seat shall not exceed 400 pounds per square inch, except for the 31st Street and 33rd Street viaducts where they are proportioned for 250 pounds per square inch.

Base plates of columns are so proportioned that the pressure upon the cut granite cap stones shall not exceed 750 pounds per square inch.

The pitch of rivets shall not be less than three diameters of the rivet, not greater than four diameters of rivet in end panels and not greater than six inches in single rows or $4\frac{1}{2}$ inches alternating, in any case. At the end of columns the pitch shall not exceed four diameters of the rivet for a length equal to twice the width of the column.

The distance from the center of a rivet hole to the rolled edge of any piece must not be less that $1\frac{1}{2}$ times the diameter of the rivet and in no case less than $1\frac{1}{4}$ inches and not greater than 8 times the thickness of the thinnest plate.

Rivet grip shall not exceed five diameters of the rivet.

Countersunk rivets shall be assumed to have $\frac{3}{4}$ of the value of rivets having full heads.

The compression flanges of plate girders have been designed of same section as that determined for tension flanges.

Columns and girders shall not be spliced without the approval of the Engineers unless so shown on the drawings. The webs of plate girders must be spliced at all joints by a plate or plates on each side of the web.

Due account must be taken of the fact that the girders under and east of 7th Avenue have been so designed that $\frac{1}{8}$ of their web section is included in the area of the flange.

The abutting joints of columns which are spliced must be milled, and the splices must be symmetrical and be sufficient to maintain the parts accurately in contact and against tendency to displacement.

If it be required to splice flanges of girders, such splices must develop the full strength of the abutting sections.

Workmanship.—All riveted work shall be sub-punched and reamed $\frac{1}{16}$ of an inch larger than the diameter of the cold rivet, and when the pieces forming one built member are put together the holes must be truly opposite, and any burr due to reaming must be removed. Material more than $\frac{3}{4}$ of an inch thick shall be drilled.

All holes for field connections shall be accurately drilled to a template or reamed while the connecting parts are temporarily put together.

Rivets shall be $\frac{3}{4}$ and $\frac{7}{8}$ of one inch in diameter, as required by drawings. They shall have full hemispherical heads except where countersunk, and shall completely fill the holes and be true and perfect in every way.

Any loose, burnt or otherwise defective rivets discovered at any stage of the work shall be cut out and replaced by the Contractor.

All shop rivets shall be machine driven.

Countersunk rivet heads shall be chipped where required by the drawings.

All riveted work shall be straight, free from open joints, and present a neat appearance. Deformity will be cause for rejection.

All rolled shapes shall be straight and true to section. Deformity will be cause for rejection.

All stiffeners on girders shall be fitted to bear against top and bottom flanges, including fillers. All end stiffeners of girders must be in a true plane and square to the flange of the girder, unless otherwise shown.

Where sole plates are used on girders they must be bolted firmly to flange angles before end stiffeners are fitted.

The foot of all columns and the tops of columns where shown on or required by drawings shall be faced at right angles to the axis of the column. This facing shall be done after connecting knees, gussets and base angles or cap angles are riveted to the column, and the connecting parts shall be placed so carefully that after facing they shall have a perfect bearing.

Shoes and cap plates shall be perfectly straight.

Where noted on drawings girders which frame between columns (and all other girders similarly marked) shall be made of the exact length required by the drawings or their ends must be faced true and square.

Girder sole plates to be planed on bottom where required by drawings.

Bearing plates on masonry to be perfectly straight and true.

Tops of cast iron pedestals shall be planed true.

No material that has been damaged in handling will be accepted in any part of the work. All workmanship shall be first class in every respect and satisfactory to the Engineers.

Quality of Material.—Rolled Steel.—The steel is to be made by the open hearth process, and the finished product must be free from injurious defects and be straight and true to section, with smooth surface and clean edges.

All steel except that for rivets shall be medium steel.

If made by the acid process it shall not contain more than 0.08% phosphorus, and if made by the basic process not over 0.06% phosphorus.

Steel shall have an ultimate tensile strength of from 60 000 to 70 000 pounds per square inch; an elastic limit at least one-half of the ultimate strength, and a minimum elongation in 8 inches of 22%.

The elastic limit of the steel is to be determined by the drop of the beam or halt in the gauge of the testing machine.

Rivet steel shall not contain more than 0.04% phosphorus, not over 0.05% sulphur and not more than 0.05% manganese.

Rivet steel shall have an ultimate tensile strength of from 48 000 to 56 000 pounds per square inch, an elastic limit not less than 28 000 lbs. per square inch, a minimum elongation in 8 inches of 28%, with an average reduction in area of about 56%.

A variation in cross-section or weight of material of more than 2½% from that specified may be cause for rejection.

Certified ladle analysis will be required of all heats, free of charge. The Engineers may have check analysis made of drillings from the finished material, which shall determine the acceptance of the material. The limits of phosphorus so found shall not exceed the specified limits by more than 25%. Samples for chemical analysis to be taken from each heat and from drillings of the finished product.

Every finished plate, bar or shape shall be stamped with a number identifying the heat and plainly marked.

Tensile and bending tests shall be made on test pieces cut from the finished material representing each heat.

The test piece shall be a manufacturers' standard test piece of 8 inches gauged length, of full thickness of material under test.

Steel shall bend cold 180 degrees around a diameter equal to the thickness of the specimen tested without showing cracks on the outside of the bent portion.

Rivet steel shall bend cold 180 degrees flat on itself without showing cracks on the outside of the bent portion.

Rivets cut out from work in which they have been driven shall show a tough, silky structure with no crystalline appearance. Every steel plate, bar or shape must be capable of standing a drifting test by punching a $\frac{7}{8}$-inch hole $1\frac{1}{4}$ inches from the edge and enlarging this hole to $1\frac{1}{2}$ times its original diameter without cracking the metal.

The fracture of all steel must be silky and have no crystalline appearance.

The Contractor shall furnish free of charge prepared specimens for testing, the use of a testing machine satisfactory to the Engineers and all facilities and necessary assistance, for making the tests.

All material shall be new and free from rust when received at the shop and there protected against undue damage to shape or surface.

No material shall be assembled for punching and reaming before it has been inspected by the duly authorized shop inspector.

Paint shall not be applied to any surface until it has been inspected.

The Contractor for the steel work shall furnish facilities for inspection and testing of material and workmanship at any time during shop hours for the duly authorized representatives of the Engineers.

Cast Iron.—All castings shall be of tough, gray iron, free from injurious cold shuts or blow holes, true to pattern and of workmanlike finish. Test bars one inch square, loaded in middle between supports 12 inches apart, shall bear 2,500 pounds or over, and deflect 0.15 of an inch before rupture.

Painting.—As soon as the work is riveted in the shop and before being exposed to the weather, it shall be thoroughly cleaned and given a thorough coating of Graphite paint or "Tockolith."

This coating shall be worked into all joints and crevices.

Surfaces which are to be riveted together shall be painted before assembling with one good coat of above paint. Any parts which are to be riveted together in erection shall receive two coats of paint before leaving the shop.

All rolled shapes except those to be used in the concrete steel floor plates shall be painted as provided herein for riveted work.

All of the steel beams of the concrete steel floor plates which are to be imbedded in concrete shall be thoroughly cleaned and shipped without painting.

All the plate girders in the work east of 7th Avenue which are to be encased in concrete shall be thoroughly cleaned and painted with

one coat of "Tockolith," except that on their bottom flanges either Graphite paint or "Tockolith" shall be used as provided herein for all other steel work.

Graphite paint to be used on the work shall be Dixon's Silica Graphite Paint, manufactured by Joseph Dixon Crucible Company, Jersey City, N. J., or other graphite paint of equal quality and satisfactory to the Engineers.

"Tockolith" to be used on the work shall be the Portland Cement paint of that name known as "Marine Tockolith," manufactured by Toch Brothers of New York City.

The vehicle of the above graphite paint shall be boiled linseed oil. No adulterated or thinner oils shall be used. The boiled linseed oil must be absolutely pure, containing no matters volatile at Two hundred (200) degrees F. in a current of hydrogen; shall not contain any resin or manganese and shall be perfectly clear upon delivery, and on standing no deposit should form, providing the oil be kept at a temperature of forty-five (45) degrees F. The film left after flowing the oil over glass and allowing it to drain in a vertical position, must be dry to the touch after twenty-four (24) hours.

No painting shall be done in wet or freezing weather and no paint shall be applied to damp surfaces or surfaces which are not thoroughly clean or dry.

APPENDIX C.

Station Building Steel.

General Description.—These specifications relate to manufacture and delivery of material required for the steel frame of the New York Passenger Station Building for the New York Terminal.

This building is a structure of the "cage" construction type, founded on masonry piers at and below track level, approximately 40 feet below the average elevation of the curbs of the surrounding streets and avenues.

The sidewalk beams along Seventh and Eighth Avenues are included in this contract, but the sidewalk beams and other structures outside of the building under Thirty-first and Thirty-third Streets are not included in this contract. Provision must be made, however, for the attachment of the parts of the Thirty-first and Thirty-third Street bridges to the building structure.

Loads.—In determining the weight of the structures for the purpose of calculating strains the weight of masonry floors has been assumed at 140 pounds per cubic foot. The structure has been designed for the dead load plus the following live loads:

Table of Live Loads.—Mezzanine Floors.—Waiting Rooms and Concourse, 150 lbs. per square foot.

All other floors below street, 300 lbs. per square foot.

Street Floors.—The entire Street Floor, 150 lbs. per square foot.

Sidewalks, 300 lbs. per square foot.

Upper Floors.—Floors of kitchen and second story shops, 150 lbs. per square foot.

All other floors above street, 100 lbs. per square foot.

Roofs.—Roofs which pitch more than 20 degrees, 30 lbs. per square foot.

All other roofs, 50 lbs. per square foot.

Proportion of Parts.—The various parts of the structures are proportioned to sustain the following unit strains given in pounds per square inch:

Tension = 16,000 lbs. (net section).

Tension in Riveted Sections = 14,000 lbs. (net section).

For determining the net sections, rivet holes are assumed to be of $\frac{1}{8}$ inch greater diameter than the cold rivets.

$$\text{Compression} = 15{,}200 \text{ lbs.} - 58\,\frac{l}{r}.$$

Wherein l is the length of the member in inches and r is the least radius of gyration in inches.

No column or strut shall have a length greater than 120 times its least radius of gyration.

Shear in webs of plate girders (gross section).	9,000 lbs.
Rivet bearing	20,000 "
Shearing strain on shop rivets shall not exceed	10,000 "
Shearing strain on field rivets shall not exceed	8,000 "

All girders shall have a sufficient number of end stiffeners to transmit vertical shear into the web, and all rivets in these end stiffeners shall be counted upon to take this vertical shear. Intermediate stiffeners shall be used at points of superimposed concentrated loading. Rivet pitch for stiffeners shall not exceed $4\frac{1}{2}$ inches. Stiffeners shall be used on all girders, the webs of which are less in thickness than $\frac{1}{60}$ of their unsupported depth, also on all girders the webs of which sustain a greater load than is allowed by the following formula:

$$P = \frac{11{,}000}{1 + \frac{d^2}{3{,}000\, t^2}}$$

In which P equals allowed strain per square inch of section.
d equals unsupported depth of web in inches.
t equals thickness of web in inches.

Intermediate stiffeners shall be spaced at intervals not exceeding 120 times the thickness of the web, and in no case more than 5 ft., as shown on detail drawings.

Girders which carry masonry arches shall be provided with shelf angles as required by drawings.

In calculating shearing strains and bearing strains on web rivets of plate girders the whole of the shear acting on the side of the panel next to the support is considered to be transferred into the flange angles in a distance equal to the depth of the girder.

Where cover plates are used, one cover plate must be of full length of girder on top flange where shown or noted.

Bearing plates for beams and girders, which rest on masonry, are proportioned in accordance with the following table of bearing values on masonry:

Brick work in Portland cement mortar, 250 lbs. per sq. in.
Portland cement concrete, 208 lbs. per sq. in.

The pitch of rivets shall not be less than three diameters of the rivet, not greater than four diameters of rivet in end panels, and not greater than six inches in single rows or $4\frac{1}{2}$ inches alternating, in any case. At the end of columns the pitch shall not exceed four

diameters of the rivet for a length equal to twice the width of the columns.

The distance from the center of a rivet hole to the rolled edge of any piece must not be less than 1½ times the diameter of the rivet, and in no case less than 1¼ inches and not greater than eight times the thickness of the thinnest plate.

Rivet grip shall not exceed five diameters of rivet.

Countersunk rivets shall be assumed to have ¾ of the value of the rivets having full heads.

The compression flanges of plate girders have been designed of same section as that determined for tension flanges.

Columns and girders shall not be spliced without the approval of the Engineers unless so shown on the drawings. The webs of plate girders must be spliced at all joints by a plate or plates on each side of the web.

The abutting joints of columns which are spliced must be milled, and the splices must be symmetrical and be sufficient to maintain the parts accurately in contact and against tendency to displacement.

If it be required to splice flanges of girders, such splices must develop the full strength of the abutting sections.

Workmanship.—All riveted work shall be sub-punched and reamed $\frac{1}{16}$ of an inch larger than the diameter of the cold rivet, and when the pieces forming one built member are put together the holes must be truly opposite, and any burr due to reaming must be removed. Material more than ¾ of an inch thick shall be drilled.

All holes for field connections shall be accurately drilled to a template.

Rivets shall be ¾ and ⅞ of one inch in diameter, as required by drawings. They shall have full hemispherical heads except where countersunk, and shall completely fill the holes and be true and perfect in every way.

Any loose, burnt or otherwise defective rivets discovered at any stage of the work shall be cut out and replaced by the Contractor.

All shop rivets shall be machine driven.

Countersunk rivet heads shall be chipped where required by the drawings.

All riveted work shall be straight, free from open joints and present a neat appearance. Deformity will be cause for rejection.

All rolled shapes shall be straight and true to section. Deformity will be cause for rejection.

All stiffeners on girders shall be fitted to bear against top and bottom flanges, including fillers. All end stiffeners of girders must be in a true plane and square to the flange of the girder, unless otherwise shown.

Where sole plates are used on girders they must be bolted firmly to flange angles before end stiffeners are fitted.

The foot of all columns and the tops of columns where shown on or required by drawings shall be faced at right angles to the axis of the column. This facing shall be done after connecting knees, gussets and base angles or cap angles are riveted to the column, and the connecting parts shall be placed so carefully that after facing they shall have a perfect bearing.

Shoes and cap plates shall be perfectly straight.

Where noted on drawings, girders which frame between columns (and all other girders similarly marked) shall be made of the exact length required by the drawings or their ends must be faced true and square.

Girder sole plates to be planed on bottom, where required by drawings. Bearing plates on masonry to be perfectly straight and true.

Tops of cast iron pedestals shall be planed true.

No material that has been damaged in handling will be accepted in any part of the work. All workmanship shall be first class in every respect and satisfactory to the Engineers.

Quality of Material.—Rolled Steel.—The steel is to be made by the open hearth process, and the finished product must be free from injurious defects and be straight and true to section, with smooth surface and clean edges.

All steel except that for rivets shall be medium steel.

If made by the acid process it shall not contain more than 0.08% phosphorus, and if made by the basic process not over 0.06% phosphorus.

Steel shall have an ultimate tensile strength of from 60,000 to 70,000 pounds per square inch; an elastic limit at least one-half of the ultimate strength, and a minimum elongation in 8 inches of 22%.

The elastic limit of the steel is to be determined by the drop of the beam or halt in the gauge of the testing machine.

Rivet steel shall not contain more than 0.04% phosphorus, not over 0.05% sulphur and not more than 0.5% manganese.

Rivet steel shall have an ultimate tensile strength of from 48,000 to 56,000 pounds per square inch, an elastic limit not less than 28,000 pounds per square inch, a minimum elongation in 8 inches of 28%, with an average reduction in area of about 56%.

A variation in cross-section or weight of material of more than $2\frac{1}{2}$% from that specified may be cause for rejection.

Certified ladle analysis will be required of all heats, free of charge. The Engineers may have check analysis made of drillings from the finished material, which shall determine the acceptance of the material. The limits of phosphorus so found shall not exceed the speci-

fied limits by more than 25%. Samples for chemical analysis to be taken from each heat and from drillings of the finished product.

Every finished plate, bar or shape shall be stamped with a number identifying the heat and plainly marked.

Tensile and bending tests shall be made on test pieces cut from the finished material representing each heat.

The test piece shall be a manufacturers' standard test piece of 8 inches gauged length, of full thickness of material under test.

Steel shall bend cold 180 degrees around a diameter equal to the thickness of the specimen tested without showing cracks on the outside of the bent portion.

Rivet steel shall bend cold 180 degrees flat on itself without showing cracks on the outside of the bent portion.

Rivets cut out from work in which they have been driven shall show a tough, silky structure with no crystalline appearance. Every steel plate, bar or shape must be capable of standing a drifting test by punching a $\frac{7}{8}$ inch hole $1\frac{1}{4}$ inches from the edge and enlarging this hole to $1\frac{1}{2}$ times its original diameter without cracking the metal.

The fracture of all steel must be silky and have no crystalline appearance.

The Contractor shall furnish free of charge prepared specimens for testing, the use of a testing machine satisfactory to the Engineers and all facilities and necessary assistance for making the tests.

All material shall be new and free from rust when received at the shop and there protected against undue damage to shape or surface.

No material shall be assembled for punching and reaming before it has been inspected by the duly authorized shop inspector.

Paint shall not be applied to any surface until it has been inspected.

The Contractor for the steel work shall furnish facilities for inspection and testing of material and workmanship at any time during shop hours for the duly authorized representatives of the Engineers.

Cast Iron.—All castings shall be of tough, gray iron, free from injurious cold shuts or blow holes, true to pattern and of workmanlike finish. The test bars one inch square, loaded in middle between supports 12 inches apart, shall bear 2,500 pounds or over, and deflect 0.15 of an inch before rupture.

Painting.—As soon as the work is riveted in the shop and before being exposed to the weather, it shall be thoroughly cleaned and given a thorough coating of paint.

This coating shall be worked into all joints and crevices.

Surfaces which are to be riveted together shall be painted before assembling with one good coat of paint. Any parts which are to be riveted together in erection shall receive two coats of paint before leaving the shop.

All rolled shapes shall be painted as provided herein for riveted work.

All paint for this work shall be made in accordance with the following specifications:

Pigment	15%	by weight.
Linseed Oil and Driers....................	75%	" "
Mineral Oil not volatile at 212° Fahrenheit..	5%	" "
Volatile Material at or below 212° Fahrenheit	5%	" "

The pigment must consist of 55 per cent. lamp black and 45 per cent. white lead, a variation of not more than two per cent. either way in any of the constituents being allowed, and should the variation exceed the two per cent. above mentioned, the paint will be rejected.

It will be noted that the paint above specified is supposed to be ready for spreading when received, and it must be so well ground, and the materials used of such a kind, that it will spread well under fair usage in the hands of the painters. The japans must be of such a kind that the paint will dry over night, under average conditions of weather, so that it may be second coated without difficulty. The lamp black and white lead must be pure and of good quality, and free from admixtures of other materials.

A sample gallon of the paint must be furnished and approved before the paint is used on the work. Samples of not less than one pint will be taken at random by the duly authorized inspector from each of ten barrels and will be sent to Charles B. Dudley, Chemist, Altoona, Pa., in "Sample for Test" box and can accompanied by "Sample for Test" tag properly filled out, and paint must not be used until report of test is received. Care will be taken to secure a well mixed sample.

The boiled linseed oil must be absolutely pure, containing no matters volatile at two hundred (200) degrees F. in a current of hydrogen; shall not contain any resin or manganese and shall be perfectly clear upon delivery, and on standing no deposit should form, providing the oil be kept at a temperature of forty-five (45) degrees F. The film left after flowing the oil over glass and allowing to drain in a vertical position, must be dry to the touch after twenty-four (24) hours.

No painting shall be done in wet or freezing weather and no paint shall be applied to damp surfaces or surfaces which are not thoroughly clean and dry.

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PAPERS AND DISCUSSIONS

THE NEW YORK TUNNEL EXTENSION OF THE PENNSYLVANIA RAILROAD. STATION CONSTRUCTION, ROAD, TRACK, YARD EQUIPMENT, ELECTRIC TRACTION, AND LOCOMOTIVES.

By George Gibbs, M. Am. Soc. C. E.

To be Presented October 18th, 1911.

Introduction.

The purpose of this paper is to describe concisely the New York Tunnel Extension of The Pennsylvania Railroad, and to record the methods followed in the design and execution of the portion of the work entrusted to the writer's department.

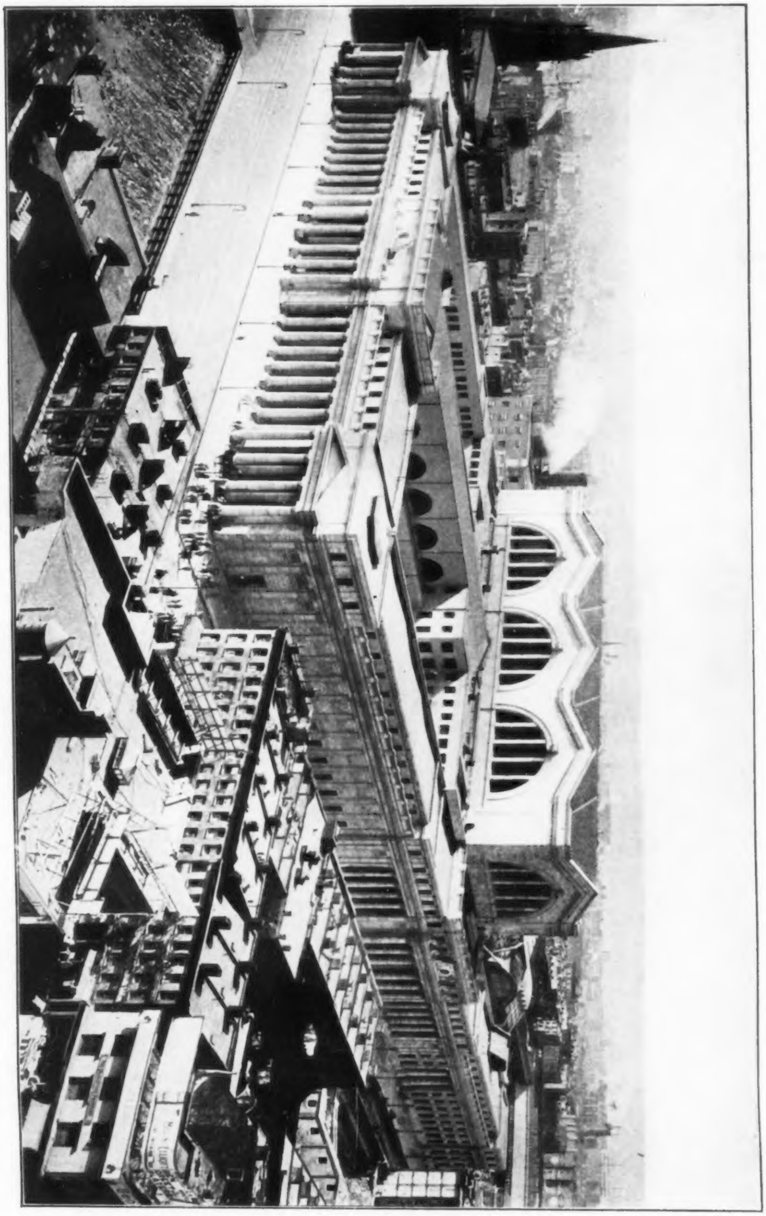
An outline of the entire terminal project is given in the paper* by Brig.-Gen. Charles W. Raymond, M. Am. Soc. C. E., Chairman of the Board of Engineers, in which it is shown that the construction was carried on under four general divisions, namely:

(1) The Meadows Division, under Alexander C. Shand, Chief Engineer, comprised the construction of an interchange yard near Newark, termed the "Manhattan Transfer," and a double-track railroad from the yard to Bergen Hill. The work of this division included the laying of all tracks ready for their equipment with signals and traction apparatus.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 1.

Bird's-eye View of Station Building from 33d Street and Seventh Avenue.



BIRD'S-EYE VIEW OF STATION BUILDING FROM 33D STREET AND SEVENTH AVENUE.

(2) The North River Division, under Charles M. Jacobs, M. Am. Soc. C. E., Chief Engineer, comprised the driving of two tunnels under Bergen Hill and the North River to Tenth Avenue, Borough of Manhattan; also the excavation and retaining-wall work in that portion of the Terminal yard west of and under Ninth Avenue.

(3) The East River Division, under Alfred Noble, Past-President, Am. Soc. C. E., Chief Engineer, comprised the excavation of the main station yard to sub-grade; the building of marginal retaining walls from Ninth Avenue to the west side of Seventh Avenue; the construction of the cross-town tunnels under 32d and 33d Streets, beginning with the normal tunnel sections between Seventh and Sixth Avenues, and continuing as four separate tunnels to and under the East River and to the Long Island City portals; the construction of the approaches, and of Sunnyside Yard, including the grading and viaducts; also, in part, the main yard buildings.

(4) The Division of the writer, Chief Engineer of Electric Traction and Terminal Station Construction, comprised the design and construction of the structures and facilities hereinafter described.

Because of the great variety of subjects involved, it has been found necessary to treat each quite generally, referring only in some detail to special features, but leaving the complete description of any and all features for other engineers. For a full understanding of the project, as it relates to railroad operation, it has been necessary to repeat certain information given in other papers, but, as far as possible, the description has been confined to the work included in the writer's Division.

Terminal Railroad.

The New York Tunnel Extension, undertaken as a separate project, starts at a connection with the New York Division of the Pennsylvania Railroad, at Manhattan Transfer, one mile east of Newark, N. J. It includes a transfer yard at that point, a double-track elevated line across the Hackensack Meadows, two tunnels under Bergen Hill and the North River to the main Station yard centrally located on Manhattan Island, thence, as a four-track railroad, across the City and under the East River to and including a very large storage yard in Long Island City, and a connection with the Long Island Railroad in that yard. Table 1 contains the general data as to the location and physical characteristics of the railroad.

TABLE 1.—SECTIONS OF TUNNEL EXTENSION RAILROAD.

Section.	Termini of sections.	Character of line.	Length, in miles.	Main Running Tracks.			Total mileage of main and yard tracks.
				Number.	Maximum curve.	Maximum adverse grade.	
Manhattan Transfer.	5th Street, Harrison, to east end of bridge over N. Y. Div.	Passenger transfer and power interchange yard	1.57	4	3°15′	0.5%	17.79
Meadows	East end of bridge over N. Y. Div. to Bergen Hill Portal	Main line embankment	4.83	2	1°54′	0.3%	9.79
North River	Bergen Hill Portal to Tenth Avenue Portal	Land and river tunnel section	2.53	2	0°30′	1.93%	5.06
Station Yard	Tenth Avenue Portal to main section near Sixth Avenue	Station yard	0.57	21	10°00′	0.4%	15.62
Cross-town Tunnels	Sixth Avenue to First Avenue Shafts	Two twin tunnels	0.89	4	1°30′	1.50%	3.56
East River	First Avenue Shafts to Long Island City Portals	Four tube tunnels	1.47	4	1°30′	1.50%	5.89
Sunnyside	Tunnel Portals to Laurel Hill Avenue	Storage and cleaning yard	1.45	8	2°42′	1.50%	36.81
			13.31				94.52

FIG. 1.—STATION YARD, LOOKING TOWARD NINTH AVENUE FROM THE WEST SIDE OF THE POST OFFICE.

FIG. 2.—STATION YARD, BETWEEN NINTH AND TENTH AVENUES, LOOKING WESTWARD.

The actual work of planning this railroad began in the early part of 1902, and continued until its completion, practically in all details, eight years thereafter. Regular Long Island Railroad train service was opened on September 8th, 1910, and the Pennsylvania Railroad service on November 27th, 1910.

The primary idea of the Terminal Railroad was to provide an all-rail line to a centrally located station in New York City, replacing the existing terminal in Jersey City, which was reached from New York only by ferries. Incidentally, it was to include a number of allied improvements intimately related to the Tunnel Extension project and with a direct bearing on its design. These were:

1st.—Improvements on the Long Island Railroad, a controlled property, by which its main terminal would be shifted from Long Island City to the New York Station, and would include an entire reconstruction of its main line to Jamaica, with four-tracking, electrification, and the elimination of grade crossings.

2d.—To open the residential sections of Long Island to the population of the thickly settled Borough of Manhattan, and to offer to Newark (a city of 347 000 inhabitants) and other populous towns in New Jersey direct and quick access to the resorts on the ocean beaches.

3d.—A direct rail connection *via* the Long Island Railroad to the West and South from the heart of Brooklyn and Queens Boroughs of New York City, containing 1 900 000 people, growing rapidly, and with ample area to accommodate a very large population.

4th.—The project (not yet carried into execution) of an all-rail connection for the New England States, through New York City, to the South and West for passenger service. This will be accomplished by building the "New York Connecting Railway" from Sunnyside Yard, on the New York Terminal Railroad, to Port Morris, on the New York, New Haven and Hartford Railroad, bridging the East River at Hell Gate.

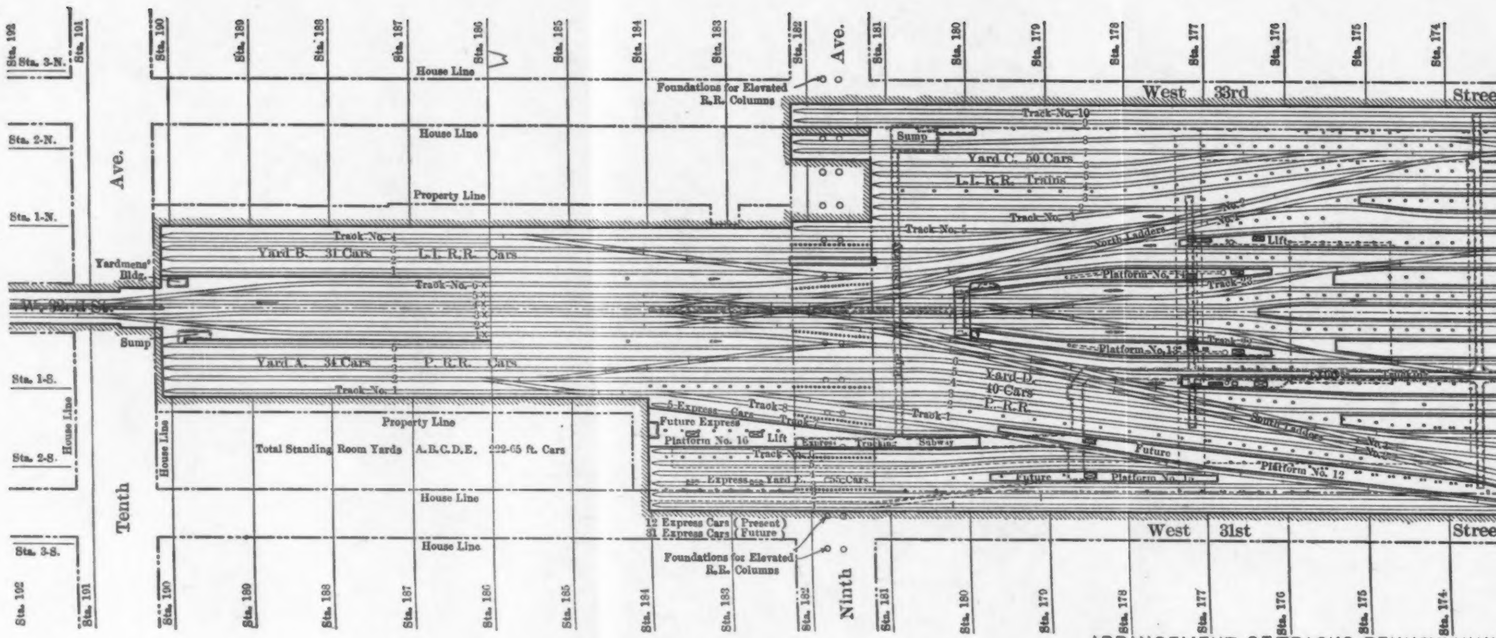
5th.—A down-town passenger terminal in the City of New York, secured by the electrification of the present New York Division tracks from Newark to Jersey City, and at that point connecting with the Hudson and Manhattan Railroad Company's line *via* tunnels under the North River to Church and Cortlandt Streets, New York City.

From the map* it is seen that the Station of this new railroad is centrally located, close to Broadway, the main city artery, and will serve the combined interests of business and residence on Manhattan Island. The district south of Central Park (59th Street) is being rapidly given over to business, amusements, and transient dwelling, and is the center for the moving throngs who require a transportation service. The Station is also central between the down-town business and the up-town residence districts.

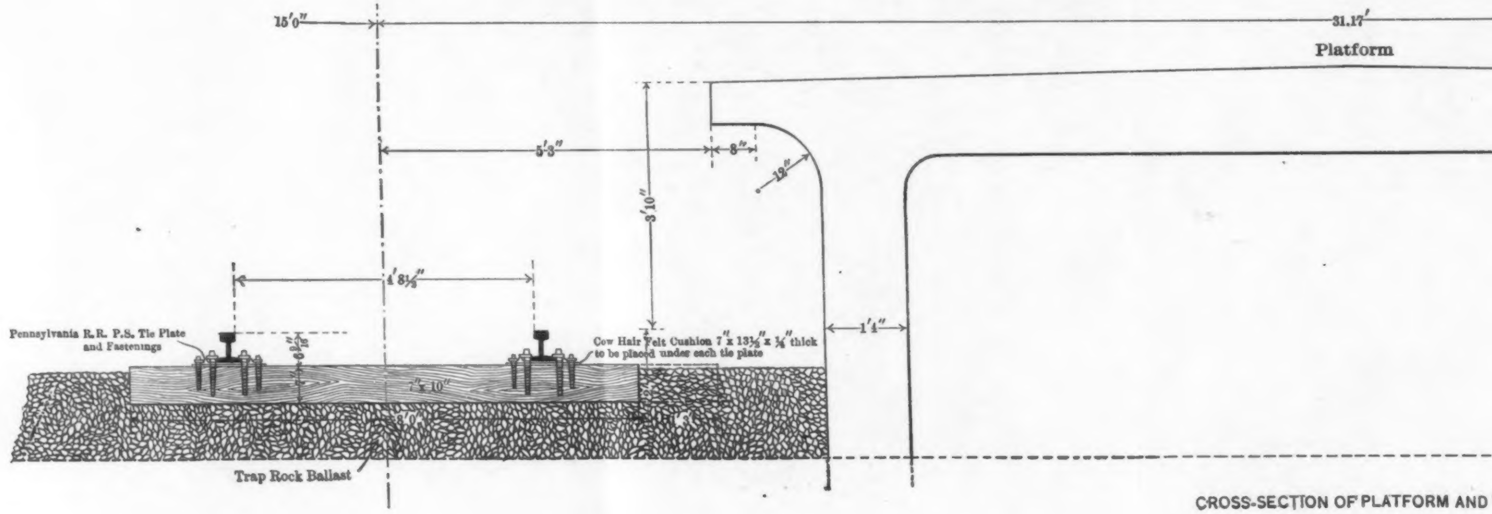
Time Saving.—The increase in convenience and the actual time saving secured to the traveling public, by the relocation of the main terminal of the great railway system which undertook this improvement, deserve recognition. Prior to undertaking the comprehensive scheme of rearrangement of and additions to its New York terminals, the Pennsylvania Railroad Company's station was at Jersey City, separated from the actual destination of its passengers by a wide river. The inconvenience of this arrangement was lessened as far as practicable by establishing a number of ferry lines to various points, along the water-front of Manhattan Island and to Brooklyn, between which fast and well-appointed boats were run at frequent intervals; but, at the best, transhipment from the trains to the city, across the river, consumed from 15 to 20 min. in good weather and about twice as much when foggy. From the marginal street along the river to the business or hotel districts, meant a further time of from 10 to 30 min., by walking or by car, through crowded city streets. Passengers to Brooklyn were obliged to make either a long ferry trip around the lower end of Manhattan Island, or a journey across two rivers and the city, with at least four transfers of conveyance, and the consequent loss of time, and inconvenience in arranging for baggage connections.

The Long Island Railroad had its main terminal in Long Island City, 10 min. by ferry to the 34th Street water-front, and 30 min. by ferry to the down-town district. Its Brooklyn terminal, at Flatbush and Atlantic Avenues, was reached by a steam railroad on the surface of a city street and subject to the delay and danger of operation under such circumstances; thence the traveler was carried into the down-town business portion of New York by surface or elevated car lines, *via* Brooklyn Bridge, and later by the subway.

* *Transactions*, Am. Soc. C. E., Vol. LXVIII, Fig. 1, p. 5.

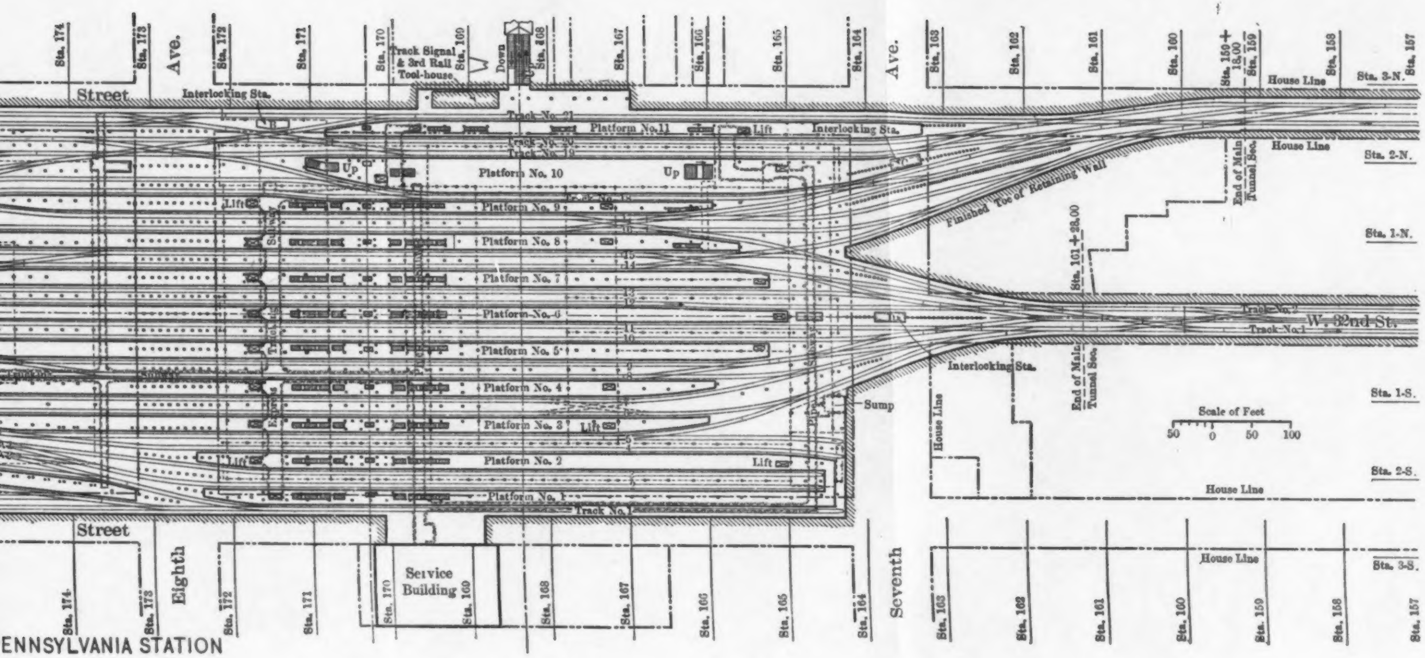


ARRANGEMENT OF TRACKS, PENNSYLVANI

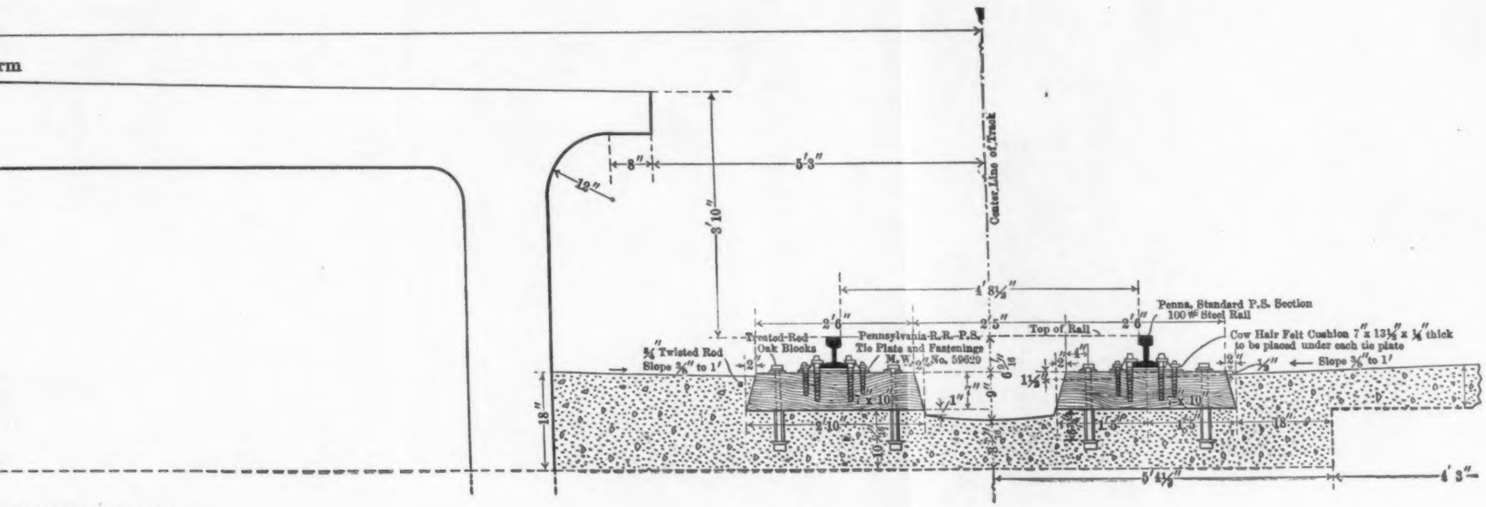


CROSS-SECTION OF PLATFORM AND T

STATION YARD



ENNSYLVANIA STATION



LATFORM AND TRACK

N YARD

Under the new conditions, briefly alluded to in this introduction, the main terminal of the Pennsylvania Railroad, used also by the Long Island Railroad, is in the heart of the greater City of New York, and is reached both from the east and west by modern electric railway lines, free from grade crossings, and suitable for high speed and frequent service. The down-town business district is reached from the west by rapid-transit service from the transfer station near Newark, and from the east by direct subway connection at Flatbush Avenue.

The time saving and convenience secured to the public by these improvements are evident. Under the new train schedules, Pennsylvania trains arrive at the New York Station in the time they formerly consumed in reaching Jersey City and lower New York, and Long Island trains in shorter time than that taken to reach Long Island City. In other words, passengers on either railway destined to the center of New York City, say Broadway and 34th Street, now save 30 min. over that required, under the best conditions, by former methods of transportation.

The collection of statistics, on which to base the required capacity of tunnels and station, was early begun by committees of the operating officers of the Pennsylvania Railroad, who formulated reports and made recommendations to the President, fixing the general characteristics of the entire project on a well-balanced plan, in which the present travel and anticipated growth for years in the future could be cared for without undue congestion of any feature.

The maximum capacity of the tunnels was based on the adopted speed and the spacing of the signals controlling train movements. In the case of the Station yard, the tracks were laid out to accommodate the maximum tunnel capacity (including tunnels to be built in the future), and tracks were assigned to each separate movement of a determined character; the switching work, time at platforms, and turn-around movements were all calculated, and working schedules were made. From the above followed a determination of the size and character of the Station Building.

A certain plan for the operating methods, assumed by the committee in fixing the station capacity, led to the following assignments of trains. This plan, of course, is subject to modifications as the classes of trains vary in the development of the service; thus, the

following is on the assumption that all Pennsylvania trains are handled by electric locomotives; if in the future, however, multiple-unit trains should be substituted for local service to New Jersey points, the station capacity would be greatly increased, as much shifting and transfer to Sunnyside Yard would be avoided:

Approach tracks from the west...........	2	
" " " " east	4	
Speed of trains, 50 to 60 miles per hour.		
Train interval under clear signal..........	2¼	min.
Capacity of each tunnel, per hour..........	26	trains.
Maximum capacity of all tunnels, per hour..	156	"

Station Movements:	Loaded trains.	Empty drafts.
Pennsylvania Railroad, on five tracks......	14	15
Long Island Railroad, " " "	19	16
" " " suburban, on four tracks	54	0
" " " head switching......	24	0
Pennsylvania Railroad, station turn-around.	10	0
" " Sunnyside Yard drafts.	0	31
Total capacity....................	121	62

Time Allowance for Trains:

Standing at platforms, through trains:	
Inbound	7 to 8 min.
Outbound	15 " 20 "
Station turn-around:	
Local trains (tail switching)	
Minimum time to unload train........	1 min. 45 sec.
" " " load "	2 . "
" " between departure of train and next arrival on same track.	1 " 20 "
Maximum daily capacity of station, based on maximum hourly capacity.......	1 160 trains.

General Information.—Other general statistics of the Terminal Railroad are as follows:

Length of run, Manhattan Transfer to Pennsylvania Station........................	8.78 miles.
Schedule time, Manhattan Transfer to Pennsylvania Station	13 min.
Length of run, Jamaica to Pennsylvania Station	11.8 miles.

FIG. 1.—STATION YARD: LOOKING EASTWARD FROM NINTH AVENUE.

FIG. 2.—HACKENSACK PORTAL OF NORTH RIVER TUNNELS.

Schedule time, Jamaica to Pennsylvania Station	18.0 min.
Total length of Tunnel Extension Railway, Manhattan Transfer to Laurel Hill Avenue, Long Island	13.31 miles.
Total single-track length of tunnels	14.52 "
Length of Bergen Hill Tunnel section	6 050 ft.
Length of river section, North River Tunnels.	6 430 "
Length of land section, North River Tunnels.	905 "
Length of river section, East River Tunnels	3 948 "
Length of land section, East River Tunnels	3 848 "
Length of cross-town section, Sixth Avenue to East River	4 696 "
Total distance, Hackensack tunnel portal to Long Island City tunnel portals	5.55 miles.
Sunnyside Yard:	
3d Street to Laurel Hill Avenue	8 815 ft.
Extreme width	1 625 "
Area	192 acres.
Length of yard tracks (present)	25.72 miles.
Length of yard tracks (ultimate)	45.47 "
Manhattan Transfer Yard:	
Length	4 050 ft.
Width	250 "
Area	23 acres.
Length of yard tracks	11.49 miles.
Station Yard:	
Length, Tenth Avenue to Sixth Avenue	3 488 ft.
Width, net, at track level (31st to 33d Streets)	509 "
Area	28.3 acres.
Length of tracks	15.62 miles.
Pennsylvania Station:	
Length, east and west	789 ft.
Length, north and south	430 "
Area (of building at street level)	7.5 acres.
Total trackage of Terminal Railroad, present.	94.52 miles.
Total main-line trackage	44.0 "
Initial daily train service, P. R. R. trains, in and out	150
Initial daily train service, L. I. R. R. trains, in and out	200
Summer schedule, 1911, P. R. R. service, in and out	200

Summer schedule, 1911, L. I. R. R. service, in and out	250
Number of electric locomotives for 1911 service	33
Number of buildings required for all purposes.	64
One traction power-house, capacity	40 000 kw.
Four traction sub-stations, total capacity	24 000 "
One service power-house, initial boiler capacity.	2 625 h.p.
Total weight of structural steel used for entire Terminal Railroad construction	80 350 tons.
Approximate total quantity of cement used for entire Terminal Railroad construction	1 942 000 bbl.
Excavation, including that for tunnels	6 936 673 cu. yd.

Station Yard.

The main yard serving the Pennsylvania Station Building is between Seventh and Tenth Avenues, and 31st and 33d Streets, including the sub-surfaces of these avenues and streets, and the surface as well as the sub-surface of 32d Street from Seventh to Tenth Avenues. Two views of the yard are shown on Plate LXVI. It is rectangular in shape, with an irregular extension west of Ninth Avenue, and covers a net area of about 27 acres. It was excavated throughout to a depth of from 40 to 50 ft. below the original surface. The engineering considerations and the work involved in clearing buildings from the site, the excavation to sub-grade, and building the marginal retaining walls have been described by members of the staffs of the Chief Engineers of the North and East River Divisions having charge of this work.

The writer's Division was charged with the design and construction of the railway, the buildings and facilities, the permanent viaducts for supporting the streets over and around the yard, the restoration of the street surfaces, and the excavation and completion of the fan-shaped approaches to the tunnels at Seventh Avenue and eastward to the normal tunnel sections in 32d and 33d Streets. The engineering features of the viaducts and sub-surface work are fully described in the paper by George B. Francis and J. H. O'Brien, Members, Am. Soc. C. E.

The two easterly blocks, including the bed of 32d Street, are occupied by the main Station Building. West of Eighth Avenue, a plot 400 by 400 ft. has been sold to the United States Government for a general Post Office Building, now in process of erection. Easement for railway purposes has been reserved under this area, below a plane

FIG. 1.—SYSTEM OF REINFORCEMENT FOR TRUCKING SUBWAYS UNDER POST OFFICE; SIDE-WALL REMOVED FOR ENLARGEMENT OF SUBWAY.

FIG. 2.—STATION YARD, FROM 31ST STREET AND NINTH AVENUE, DURING TIME OF TRACK LAYING.

20 ft. above the tracks, subject to the reservation for the necessary supporting columns for the building, in locations fixed in the deed. West of the Post Office the area to Tenth Avenue is an open yard, bridged only by the viaducts over the side streets and Ninth Avenue. This open area may eventually be covered by marginal buildings. A plot 130 by 180 ft. has been reserved west of Ninth Avenue and at the north side of 31st Street for an Express Building, should it be desired at a later date to make the terminal a distributing point for express matter.

The sub-surfaces of Seventh, Eighth, and Ninth Avenues, and of 31st and 33d Streets, are in part occupied by the yard and in part by the retaining walls. It is of interest to state that everywhere in the yard the top of the track rails is from 9 to 23 ft. below mean high water in the harbor.

Track Plan.—The track plan is shown on the upper part of Plate LXVII. The summit of the yard is 530 ft. west of Seventh Avenue, from which point the grade falls east and west to the tunnel portals. From Tenth Avenue the grade rises eastward at the rate of 1.923% to a point midway between Tenth and Ninth Avenues, thence at the rate of 0.4% to the summit, and falls at the same rate to the Cross-town Tunnels. The track plan adopted was the result of extended study by the Terminal Committees of the Pennsylvania Railroad, and involved the making of about twenty tentative schemes in order to harmonize the operating requirements with the progressive development of the Station and Post Office Building plans. The primary aim was to balance the yard and tunnel capacity at the same maximum, having in mind the expectation that the yard should be capable of accommodating in the future the traffic from two more tunnels under the North River, making four in all, and two more under the East River, making a total of six. In doing this, certain assumptions were necessarily made as to the character of the traffic, that is, the amount of business handled by multiple-unit trains and those requiring electric locomotives, also the character of the switching operations to be done in the yard, and the amount of car storage needed.

The main approach tracks from the west fan out from the tunnel portals at Tenth Avenue into six running tracks, three for each direction, to the main switch leads at Ninth Avenue, thence by double ladders, one to the north and one to the south, into twenty-one plat-

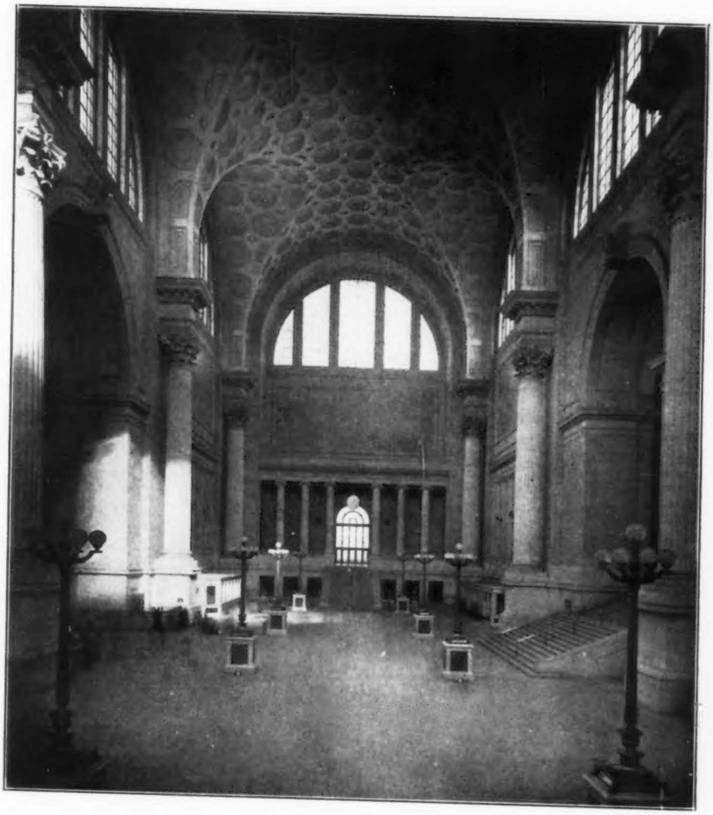
form tracks covering the full width of the yard. Fig. 1, Plate LXVIII, is a view of the yard from Ninth Avenue, looking eastward. At the east end all except five of these tracks are again gathered into two groups, one leading to the two tunnel tracks under 33d Street, and one to the two tracks under 32d Street. At the extreme southeast, five stub tracks end at Seventh Avenue, to lead in future to two tunnels under 31st Street. It has been the aim to provide double leads throughout, as far as possible, so that simultaneous parallel movements can be made in reaching the various tracks connected therewith.

The station yard is not intended to provide unlimited car storage, but, nevertheless, considerable space for this purpose is available in the corners formed by the rectangular configuration of the lot and the rectangular extension west of Ninth Avenue, as shown by Table 2.

TABLE 2.—STATISTICS, STATION YARD.

PLATFORM FACILITIES AND CAR STORAGE TRACK ROOM.		
Number of station tracks adjoining platforms		21
Length of platforms adjacent to passenger tracks		21 500 ft.
Total area of platforms		244 270 sq. ft.
Number of passenger platforms		11
" " mail "		2
" " express " (future)		4
Four passenger platforms, average	21 ft. wide	900 ft. long.
" " " "	21 ft. "	1 050 ft. "
Two " " "	16 ft. "	1 000 ft. "
One " " " (L. I., loading)	47 ft. "	745 ft. "
Two mail " "		375 ft. "
Longest platform (No. 6)		1 140 ft. "
Total length of future express platforms		1 600 ft.
Mail cars standing at tracks adjacent to platforms		25 cars.
Storage space in yard "A" (Pennsylvania turning yard)		34 "
" " " " "B" (Long Island express yard)		31 "
" " " " "C" (" " local ")		50 "
" " " " "D" (Pennsylvania turning yard)		40 "
Total yard storage capacity		181 "
GENERAL STATISTICS OF THE STATION YARD AND FACILITIES.		
Total excavation (place measurement) for the entire yard, all contracts included		2 200 000 cu. yd.
Excavation begun (Mr. Noble's contracts)		July, 1904.
Viaduct erection started		Oct., 1905.
Viaducts completed		Jan., 1910.
Number of viaduct columns		771
" " Station Building columns		650
" " Post Office Building columns		200
" " columns for other structures		16
Total length of viaducts		5 985 ft.
Weight of structural steel in viaducts		23 500 tons.

The parallel tracks are generally spaced at 31-ft. centers, where platforms are between, and at 15-ft. centers otherwise, to allow for the placing of building column supports. The total length of tracks in the station yard is about 16 miles, of which 3 miles are adjacent to platforms.



MAIN WAITING-ROOM.

Clearances.—The minimum overhead clearance in the yard between top of rail and girders is 16 ft. 2 in., the same as the clearance in the tunnels. The maximum car equipment height is 14 ft. 9 in. over all, and 15 ft. over all for locomotives, leaving a net minimum clearance of 1 ft. 2 in. between equipment and permanent overhead structures. The overhead contact rail has a clearance of 15 ft. 4 in. from the under contact surface to the top of the track rail.

The minimum clearance between sides of cars and columns between tracks is 1 ft. 6 in., whether the cars are on tangents or curves, and the minimum clearance for trucking on platforms is 5 ft., although in a few special instances it has been necessary to reduce this slightly. The clearance between the edges of the high platforms and the sides of the cars is 4 in.

Subways.—In the terminal yard area between Seventh and Ninth Avenues a comprehensive system of subways has been constructed under the tracks. They were deemed essential for the following purposes:

1st.—For housing the various piping systems to the buildings and tracks. These systems comprise very long runs, many of the pipes being of large diameter, to convey steam, air, and water, and have numerous connections to the buildings and at the track level. Proper and convenient maintenance required that all pipes should be readily accessible, which result could not be had by laying them in the ballast above the sub-grade; furthermore, because of the close spacing of the tracks, and the fact that this interspace is in many places occupied by columns, there was really no opportunity to lay out a practical pipe system in the ballast, and the buildings above the tracks were without basements in which to construct such a system.

2d.—The high station platforms (shown on the lower part of Plate LXVII), and the arrangement of the building itself, required provision for trucking baggage, mail, and express matter from one platform to the others, and this could be accomplished best by cross-trucking subways under the tracks. In places this was the only method possible, as some of the platforms are under the streets, and the baggage-rooms can be reached only by underground means.

3d.—Communication between the various important buildings, such as the Station, the Post Office, and the Express Building, could not be conveniently had at the track level or above, and therefore longitudinal trucking subways between them were necessary.

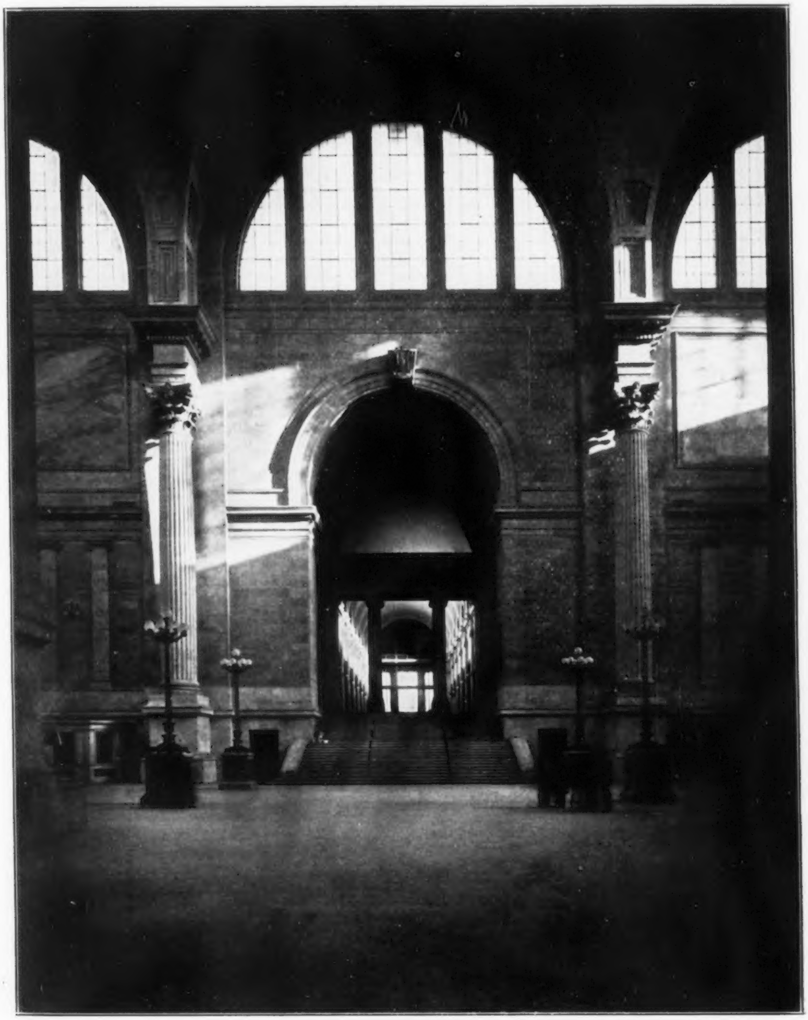
4th.—In the portion of the yard under and west of the most important signal cabin ("A"), at the intersection of the main-track ladder system, accessible space was required for the large quantity of signal apparatus, such as wiring, transformers, relays, and batteries. This space was provided by constructing a basement for the building, below the tracks, and a longitudinal subway for the conduits and instruments; from this subway all connections were conveniently made to the switches, and all apparatus was placed within easy reach.

The subways, therefore, while expensive to construct, furnish a valuable and comprehensive means of intercommunication for many important facilities, without interfering in any way with the scheme of tracks or train operation, or requiring important modifications in the buildings themselves.

As will be seen from the track plan, the subways include a main longitudinal system, from Seventh to Ninth Avenues, used in part for pipeways, in part for trucking purposes, and generally for both. This longitudinal system is intersected by cross-galleries consisting of baggage-trucking subways at each end of the Long Island platforms at the north side of the yard, a main cross-trucking subway immediately under the outgoing baggage-room in the Station Building and communicating with all lifts, and four cross pipe subways approximately 400 ft. apart, to cover the entire yard.

By the above, therefore, all platforms have intercommunication through the subways and by the lifts; the Station Building has access to the Post Office by a subway and lift reaching the operating floors of the latter building, and the same facilities will be furnished for the Express Building when erected. The building of these subways, as well as other sub-surface structures in the yard, such as under-drainage, building foundations, and conduits for power and telephones, was included in an important contract, the execution of which is recited in the paper by Messrs. Francis and O'Brien. Fig. 1, Plate LXIX, is a view under the Post Office, showing the system of reinforcement for the trucking subways; Fig. 2, Plate LXIX, is a view of the Station yard from 31st Street and Ninth Avenue, during the time of track laying.

Wall Around Yard.—The open area of the yard, to the west side of Ninth Avenue, has been permanently fenced by the construction of



MAIN WAITING-ROOM, LOOKING TOWARD ARCADE.

a marginal wall at the street level. This wall is carried on the street viaduct structure, is 6 ft. 5 in. high at the sidewalk side, and has a total length of 1 440 ft., enclosing Ninth Avenue, 31st and 33d Streets. The body of the wall is of a special gray mottled brick (in color closely resembling granite), with a base course and coping of granite. There are pilasters at intervals of about 40 ft. and paneled brickwork between.

Station Building.

The plan and engineering features of this great building, designated "Pennsylvania Station," deserve more extended description than can be given in a general paper covering so many subjects. In size, the building is unprecedented among railway stations, having a ground area of 7½ acres, about twice that of St. Peter's, in Rome, and 30% more than the area of the Palais de Justice, in Brussels (the latter being considered the largest building of the 19th century), and two and one-half times that of the New York Public Library. In length, the Station, east and west, is somewhat greater than that of the Capitol at Washington. In contents, it measures about 40 000 000 cu. ft. In plan, it is intended wholly for railway purposes and the comfort and convenience of passengers, and its design has not been subordinated to the purposes of a hotel or office building. In consonance with its legitimate purpose, however, the aim has been to make the structure a monumental gateway to the largest city in the country.

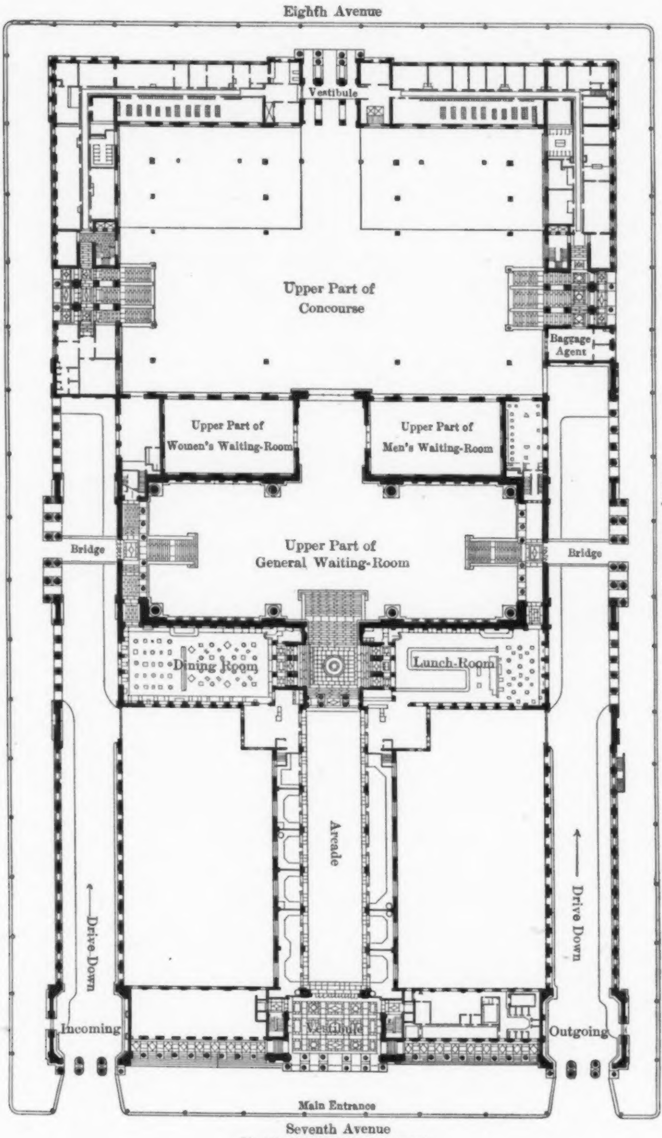
General Plan.—A passenger terminal may be located either adjoining or over the tracks served. The former plan has been generally adopted heretofore, where land was cheap, and especially because of the necessity of large open spaces in order to dispose of the smoke and steam from locomotives. In very large modern terminals, however, this plan has disadvantages in the costly property involved and in the enormous distances between the head-house facilities and the point where the passengers board the trains. Furthermore, with the advent of trains propelled by electric motors, it is entirely possible to utilize the basement of a station building for the tracks, and the levels immediately above for the facilities. Thus, when passengers arrive at the station, they are at the nearest point to the one where they embark on the train. It is true that this arrangement involves different levels, and thus the use of stairs or lifts, but, in order that a station may be in the heart of a great modern city, the depression or elevation

of the tracks is unavoidable, and stairs are a necessity, regardless of the location of the building.

The form of construction adopted for the Pennsylvania Station, therefore, is that of a bridge over the yard and platforms, the building having its main floor intermediate in level between the streets and the track platforms. The main station facilities are centrally located as regards the building itself, which may be entered from any one of its four sides, and also as regards the trains at the platforms underneath, so that the distances, which are necessarily considerable in a station of large capacity, are the least possible. It is believed that the unusual opportunities afforded for entrance and exit at the centers of the four sides, and the separation of incoming and outgoing passengers on different levels, make this building unique among the large stations of the world in the distribution of crowds without delay or confusion. These numerous means of access, moreover, should prove an important factor in building up the section of the city surrounding it, not only from one but from all sides.

Because of the location of the passenger facilities on different levels, it is difficult to show clearly by plans and cross-sections the arrangement of the building, nor can a clear description be concisely given. In general, however, the station consists of a hollow rectangle of marginal buildings surrounding the plot from the street level upward, as shown by Fig. 1; an intermediate building at the street level, starting from the middle of the Seventh Avenue façade, used for a main entrance and arcade, and continuing to its north and south axis, where it joins a high cross-structure containing the main waiting-room, with the floor one story below the street level, Fig. 2. Immediately east of the main waiting-room and on both sides of the arcade, at the street level, are located the restaurant and lunch-room. The rectangular spaces between these latter, the arcade, and the marginal buildings, are open courts, roofed over at the street level with glass skylights, for train-sheds and for driveways (Fig. 1, Plate LXXII) to the baggage-room under the arcade. These driveways communicate with the inclined interior streets, or drives, entered from the north and south ends of the Seventh Avenue front.

Continuing westward from the main waiting-room (Plates LXX and LXXI) and on the same level (one story below the streets), there are two sub-waiting-rooms, and between these and the marginal build-



PLAN AT STREET LEVEL
NEW YORK STATION

FIG. 1.

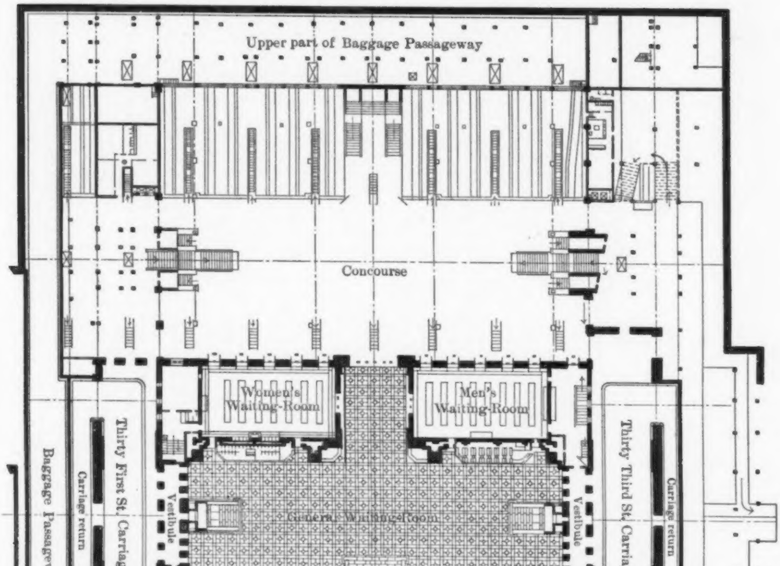
ings at the west end of the Station, the space is occupied by the main concourse (Fig. 2, Plate LXXII), a roofed-over structure of glass, containing the assembly space for outgoing passengers, prior to their admission to the stairways (Fig. 1, Plate LXXIII), leading down to the track platforms. Under this main concourse there is an exit concourse (Fig. 2, Plate LXXIII), narrower than the main one, and having stairways at either side from each platform, for incoming passengers, Fig. 3. Both these concourses connect at their respective levels with a two-deck passageway under 33d Street, the entire length of the Station between Seventh and Eighth Avenues, and designed to connect with future rapid-transit subways in either of these avenues. At present this passageway is used for intercommunication with the Long Island section of the station, elsewhere referred to, and for an entrance and exit to 34th Street.

The marginal buildings, on the Seventh Avenue front, above the street level, are used for stores, offices, and entrances to the driveways. On the side streets, as far west as the concourse, the first two stories are given up to driveway spaces, the ceiling being at the attic floor level. West of the driveways the buildings contain offices and the operating staff facilities (see Figs. 4, 5, and 6).

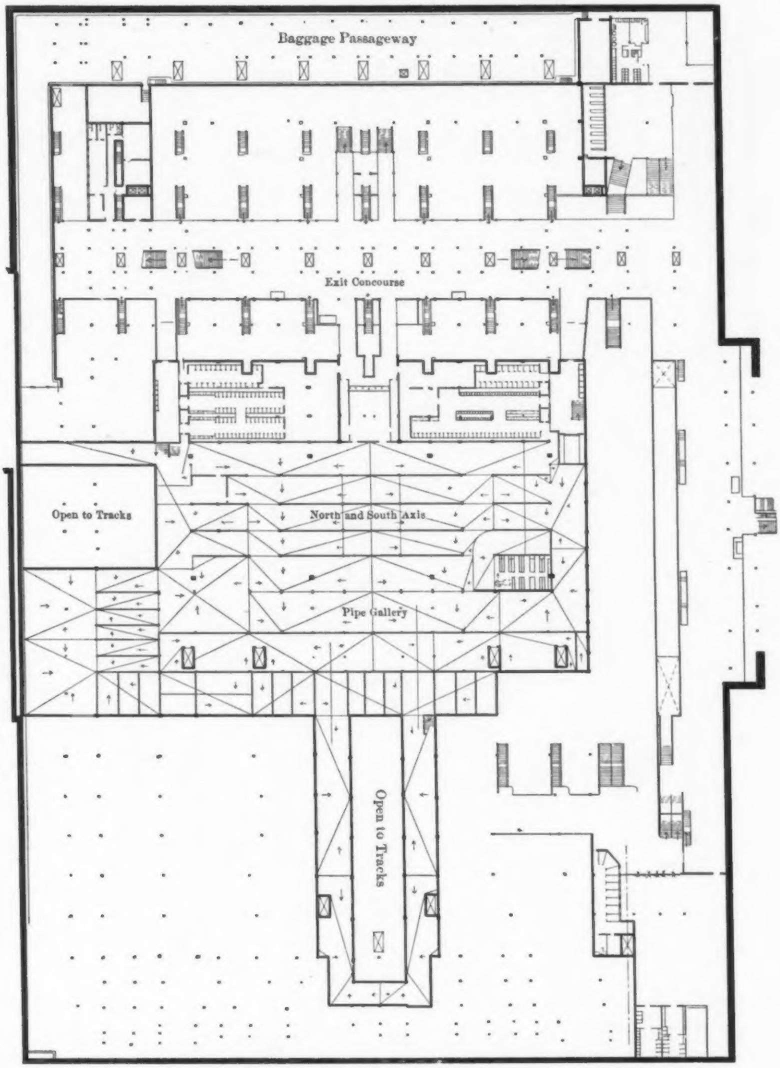
The central vestibule of the Seventh Avenue façade may be considered as the main foot-passenger entrance, leading from which is the arcade, flanked by shops on either side; thence down one floor level by the main stairway to the main waiting-room, in which are the ticket-offices and parcel-rooms, the baggage-checking booth, and other minor facilities. The waiting passenger may then proceed to the sub-waiting-rooms (smoking or non-smoking), where seats are provided; or, without retracing his steps, may enter the main concourse to the west, where the gates leading to the outgoing train platform stairs are located. Both the main hall and the concourse may be entered from two side streets, and the concourse from Eighth Avenue as well. Fig. 1, Plate LXXIV, is a view of the Eighth Avenue façade.

Carriage passengers enter the building from the south end of the Seventh Avenue front by a driveway leading down to the waiting-room and on its level, and outgoing carriage passengers leave by a similar driveway at the north side of the building. These driveways are also used for the wagon delivery of baggage to and from the baggage-room.

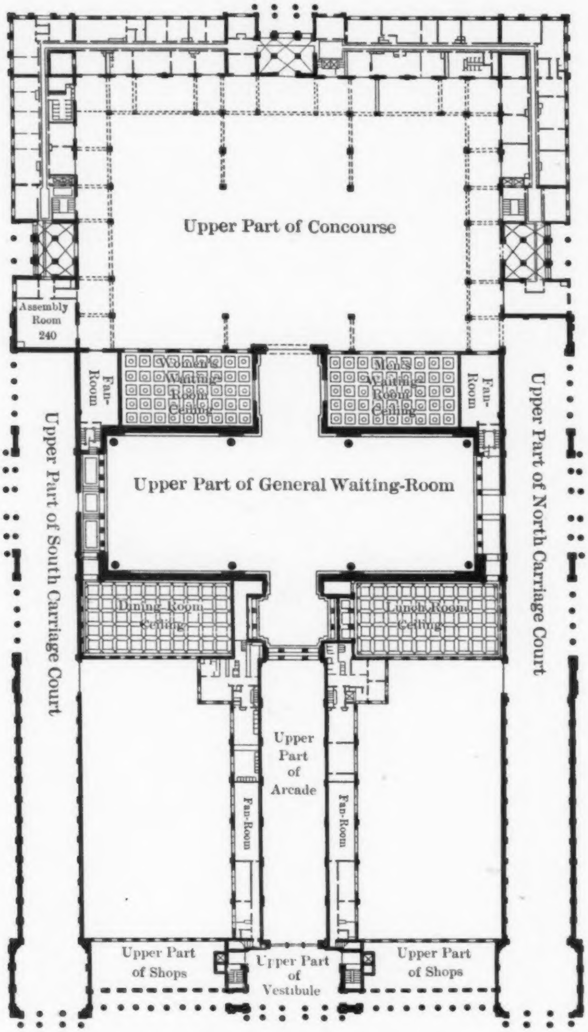
To accommodate the large number of suburban and commuter



PLAN AT WAITING ROOM LEVEL
NEW YORK STATION
Fig. 2.

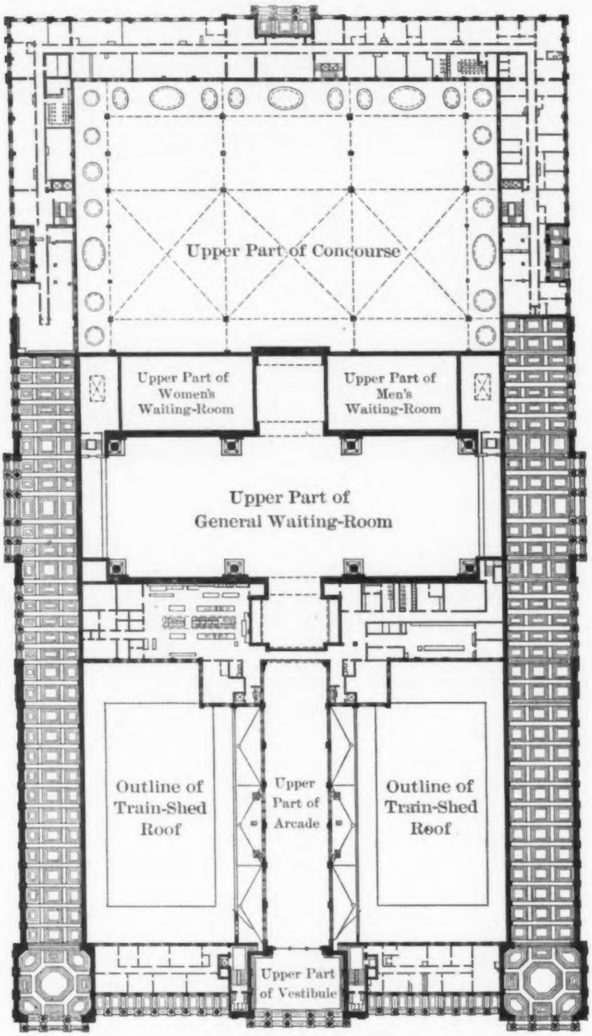


PLAN AT EXIT CONCOURSE LEVEL
NEW YORK STATION
FIG. 3.



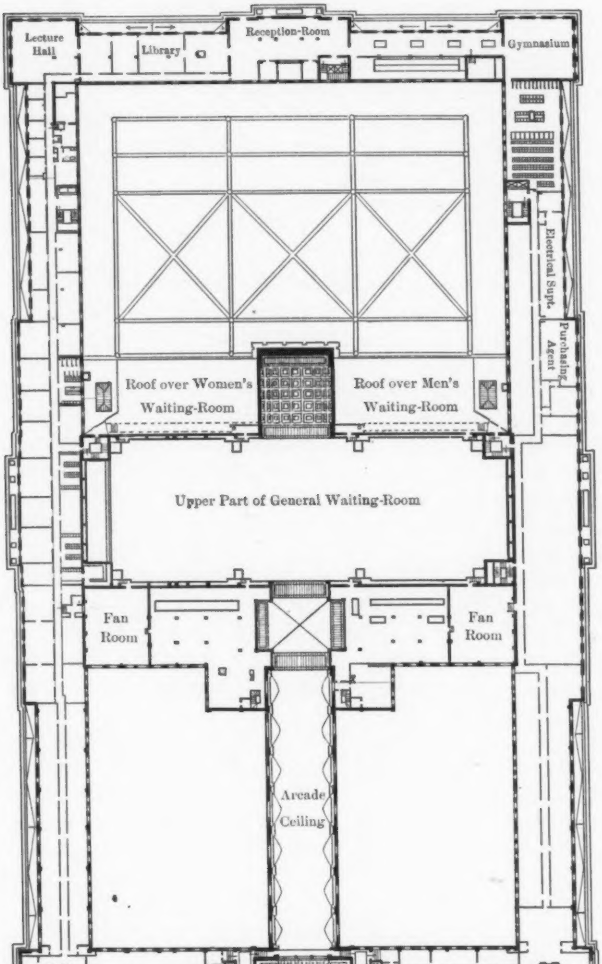
PLAN AT SECOND FLOOR LEVEL
NEW YORK STATION

Fig. 4.



PLAN AT THIRD FLOOR LEVEL
NEW YORK STATION

FIG. 5.



PLAN AT ATTIC LEVEL
NEW YORK STATION

FIG. 6.

passengers of the Long Island Railroad, a practically independent station, within the main station, has been provided. This is at the north side, under and adjoining 33d Street. It is entered at the Seventh Avenue corner, by the driveway, by a stairway from 33d Street, or by an entrance and exit from 34th Street, midway between Seventh and Eighth Avenues, and there is a separate waiting-room, with the usual facilities, a wide departure platform, and a concourse communicating with other platforms.

Architecture.—The following description of the architectural motif of the building design has been kindly furnished by Mr. W. Symmes Richardson, of Messrs. McKim, Mead, and White, the member of that firm who was especially charged with working out this problem.

"In designing the Pennsylvania Station, an attempt has been made, not only to secure operating efficiency for one of the largest railway stations in the world, but also to obtain an outward appearance expressive of its use, and of a monumental character. The problem involved was unusual, as the tracks are situated so far below the surface of the street that it was not possible to adopt any of the types of station buildings familiar in modern architecture. The exposed train-shed, with its large semicircular ends of glass, has become, during the last century, a form recognized by the layman as the railway type, and such features at the ends of the avenues of our modern cities suggest a great terminal, even to a stranger, when seen for the first time. Of such a character are the Gare de l'Est, the Gare Montparnasse, and the Gare du Nord, in Paris, the stations at Frankfort and Dresden, and, in fact, most of the principal stations of Continental Europe, as well as the splendid train-sheds of the Pennsylvania Railroad Company in Jersey City and Philadelphia.

"Not only did the architects desire to give an adequate railway expression to the exterior, but they recognized the equal importance of giving the building the appearance of a monumental gateway and entrance to one of the great metropolitan cities of the world. This idea, in their opinion, has not always received the recognition which it deserves in the solution of problems of this character.

"For inspiration, the great buildings of ancient Rome were carefully studied, and particularly such buildings as the Baths of Caracalla, of Titus, and of Diocletian, and the Basilica of Constantine, which are the greatest examples in architectural history of large roofed-in areas adapted to assemblages of people. Moreover, the conditions of modern American life, in which undertakings of great magnitude and scale are carried through, involving interests in all parts of the world, are

more nearly akin to the life of the Roman Empire than that of any other known civilization. It seemed, therefore, fitting and appropriate in every way that the type of architecture adopted should be a development from Roman models, and while the building is of necessity, on account of the requirements of its uses, different from any building known to have been previously built, its inspiration can be directly traced to the great buildings of the Roman Empire.

"To obtain the largest possible expression, simple materials have been used throughout. The exterior being entirely of granite, all unnecessary detail of ornamentation was omitted, and it has been hoped, considering the variegated character and style of the modern architecture of American cities, that in this way the monumental mass and scale of the building has been maintained in relation to its surroundings. The design is of Roman Doric, surrounded by an attic, with a colonnade along the Seventh Avenue front, and with colonnades on the other sides marking the principal entrances. To avoid monotony of effect in a building of such unusual frontage, the attic is broken into pavilions of varying heights, marking the important entrances. In the center of the rectangle, and dominating the entire structure, rises the wall of the main waiting-room, the largest room of its kind in existence. This wall is treated as a background to the buildings facing the street, and is broken simply by eight large semicircular openings of glass, each nearly 75 ft. in diameter, which light the room and give to the building, when seen from a distance, something of the railway character above referred to. Apart from the practical consideration of obtaining adequately roofed-in areas, this room was primarily created to give the exterior of the building as distinctive a railway expression as was possible, considering the limitations of the problem.

"At the north and south ends of the Seventh Avenue front are porticos leading to inclined descending driveways, forming entrances for carriages, which pass between the columns in the same way as in the Brandenburg Gate in Berlin, through which a great part of the traffic enters that city.

"The official foot entrance to the station is in the center of the Seventh Avenue front, opposite West 32d Street. This leads directly to the general waiting-room, in the center of the building, through an arcade, somewhat similar in scale and idea to the famous arcades of Milan and Naples, Italy. The main waiting-room is comparable in dimensions to the nave of St. Peter's Cathedral, in Rome. At the entrance to the waiting-room is a stairway 40 ft. wide, at the side of which is a niche containing the statue of the late A. J. Cassatt, President of the Pennsylvania Railroad, and the dominant personality in the tunnel and station project. The motif of the waiting-room design was suggested by the great halls of the baths of ancient Rome,

above referred to, and consists of eight Corinthian columns, 7 ft. in diameter and 60 ft. high, standing on pedestals, and supporting the coffered vaulted ceiling. At the north and south ends of the rooms are colonnades of single Ionic columns, 31 ft. high, directly approached by bridges over the carriage driveways, from the central entrances on West 31st and West 33d Streets, and from which ample staircases lead to the floor of the room. The sub-waiting-rooms, opening into the retiring-rooms, are proportioned to the magnitude of the central room. The connecting openings are made as large as possible, and frequently are of screens of clear glass of great dimension, permitting comprehensive perspective views, not only of splendid architectural effect, but of great assistance as a guide to the movements of the traveling public. For the interior, the architects have selected a Roman travertine stone, brought from the quarries near Tivoli, Italy. Of this stone, the exterior of the Colosseum, the Tomb of Hadrian (now the castle of St. Angelo), the Quirinal Palace, the Cathedral of St. Peter's, and nearly all the churches and most of the palaces of Rome are built. Considered purely from the structural standpoint, it is one of the finest building stones known, but its selection for this building, for which it has been imported into this country for the first time, was due principally to its beautiful, warm, sunny color, and its tendency to take a polish and improve in appearance by contact and use rather than to absorb dirt, as is the case with so many of the limestones in common use both here and abroad. The stone, moreover, has a very interesting visible structure, which, in a building of such large dimensions, tends to give a more robust character and texture than it is possible to obtain in most other materials. A color motif has been given to the room by the insertion of conventionalized maps in the six large panels below the lunette windows; these maps were painted by Mr. Jules Guerin.

"The concourse itself forms a courtyard with granite walls, enclosed by an iron and glass roof, forming intersecting barrel arches surrounded on three sides by tile domes against the walls of the building. The structural steelwork here is of an open latticed design, without ornament, the architectural effect being obtained by a careful study of the proportions and form of the structural members required. Here the architects have attempted to give to the structural steel a straightforward and adequate architectural expression, and while the design is quite different from anything yet built, it is suggestive in many ways of the train-sheds in the famous stations at Frankfort and Dresden, Germany. On the easterly side of the concourse is the continuous façade of the waiting-room, with semicircular openings, comparable in extent and scale to the Boston Public Library.

"The design, fabrication, and erection of the concourse roof introduced novel problems. It was the desire of the architects to give the structural steel a dignified expression of design, and also to obtain an

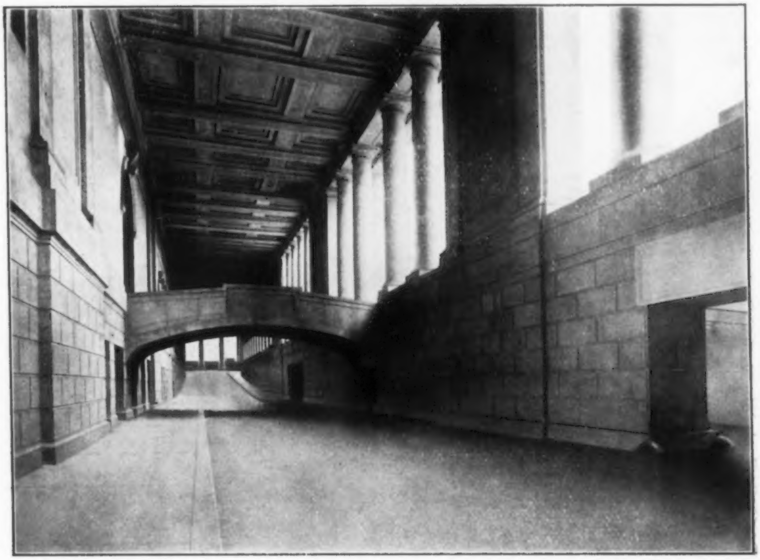


Fig. 1.—Thirty-first Street Carriage Driveway.

Fig. 2.—Main Concourse.

appropriate transition between the purely architectural lines and structural materials of the general waiting-room and adjoining rooms and the purely utilitarian and structural treatment of the railway operating features of the yards, such as the tracks, viaducts, etc., that is to say, the leading by an easy and unconscious gradation of effect from the monumental side of the Station to the utilitarian. To accomplish this result the main architectural lines of the concourse roof were first determined, namely, the location of the columns, arches, and domes, and the general height and breadth of the intersecting members; the steel engineer being then given the problem of designing a structure to conform to the architectural lines outlined, the detail being a question of good proportion and adjustment from both points of view. The type and scale of lattice work, as well as the lines and sizes of the arches, notably the variation in depth between the spring and top lines of the arches, as well as varying widths between the diagonal ribs and the vault lines, was suggested to the engineer to obtain a variety of effect and to avoid the monotony which would result in the assembling of arches of similar forms and dimensions. To obtain the expression of these architectural features in steel, necessitated the use of an excess of material over what would be required by ordinary trussing to cover the area in question, but the excess amount of material was considered justifiable to fulfill the motives above referred to. The design of this difficult piece of roofing was due to the joint efforts of the Architects, Westinghouse, Church, Kerr and Company, and Messrs. Purdy and Henderson, Engineers.

"Throughout there has been a consistent and continuous effort to maintain a unity and simplicity of design, so that the structure will count as a whole of many inter-related parts of similar scale. Ornament has been very sparingly used, and there is no attempt at decorative art, except for the color effect of the maps in the main waiting-room. The interior of the building is practically a monotone, it being the idea of the architects that a building devoted to railway purposes should be made of permanent and durable materials of simple character and capable of the easiest maintenance. The light buff of the travertine stone has formed the keynote of the color scheme for the plaster walls and ceilings, the larger ceilings having a pigment in the plaster to give a permanent stain, so that the necessity and inconvenience of repainting is reduced to a minimum. In the few places where decorative sculpture is used, such as the clocks over the main entrances, the eagles and bas-relief panels adjacent, and the large keystones on the exterior of the general waiting-room and over the arches leading from this room to the arcade and concourse, the work was placed in the hands of Mr. A. A. Weinman, a sculptor of reputation, who has given to the ornament and figures a distinct individuality appropriate to the uses of the building."

The following is a list of the principal building subdivisions, with spaces allowed for facilities:

Length of building, east and west	789 ft.
" " " north and south	430 "
General height from sidewalk	76 "
Extreme " " "	153 "
Height, interior of waiting-room	150 "
" " " dining- and lunch-rooms	32 "
" " " sub-waiting-rooms	56 "
" of concourse	100 "
" " exit concourse	11 "

	Dimensions.	Area, in square feet.
Concourse court	340 by 210 ft.	71 400
Concourse floor	475 " 125 "	60 000
Exit concourse floor	480 " 60 "	28 800
33d St. passageway, Seventh to Eighth Ave	654 " 30 "	26 500
Main waiting-room floor	300 " 110 "	33 000
Arcade	220 " 40 "	8 800
East baggage-room (T-shaped) (total area)	246 " 90 "	34 000
West baggage-room	321 " 50 "	16 000
Train-sheds (two, dimensions each)	216 " 112 "	48 384
Standing room for cabs (exit side)		Capacity 25
Lunch- and dining-rooms (two, each)	115 " 60 "	6 900
Sub-waiting-rooms (two, each)	100 " 60 "	6 000
" " " total seating capacity		700 persons
Women's retiring-room	30 " 38 "	1 140
Seventh Avenue shops (two, each)	100 " 35 "	3 500
Arcade shops (two sides, each)	184 " 76 "	2 944
Barber shop	50 " 30 "	1 500
Waiting-room ticket offices	20 windows.	2 400
Carriage driveway, 33d Street	552 by 45 "	24 840
" " 31st Street	530 " 41 "	21 730
Sidewalks around building	2 650 " 30 to 40 ft.	90 000
Women's main toilets		3 140
Men's " "		3 600
Hospital Department		1 400

FIG. 1.—CONCOURSE, SHOWING STAIRWAYS TO PLATFORMS.

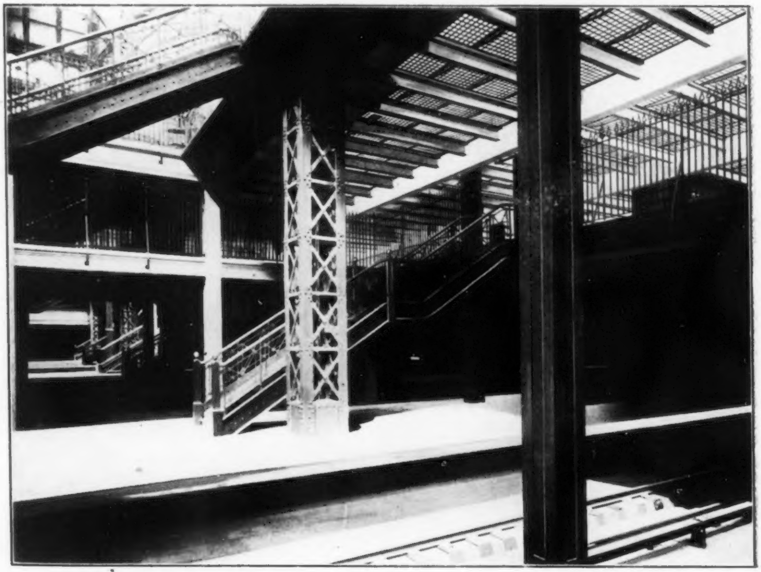


FIG. 2.—PLATFORMS AND STAIRWAY TO EXIT CONCOURSE.

	Area, in square feet.
Police Department	827
Funeral-rooms	600
Parcel-room	2 780
Office space, 1st floor, west of waiting-room	15 850
" " 2d floor, west of waiting-room	20 270
" " 3d floor, west of waiting-room	31 500
" " Seventh Avenue, front, two floors	21 500
Y. M. C. A., attic floor	16 800
Bunk-rooms, " "	10 200
Unassigned space, attic floor	27 800
Kitchen and store-rooms	15 000
Ticket offices, men's waiting-room	2 344
Ticket offices, Long Island section	2 870

	No.
Passenger elevators to track platforms	11
Baggage lifts	21
Stairways to track platforms	48
News-stands	8
Boot-black stands	7
Clocks	30

General Constructive Features.

It has been the aim to make the building thoroughly fire-proof. Practically no combustible materials have been used in the construction or furnishings of the public rooms, baggage-rooms, or spaces devoted to railway uses below the street level; wood trim is used only for the offices.

Steelwork.—The designing of the building framework is described in detail in the paper by Messrs. Francis and O'Brien. It called for the fabrication of about 27 000 tons of structural steel, in large part of special design, there being little duplication for the different sections of the building or on the different floors.

Granite.—The exterior is of curtain-wall construction, with a granite face. The granite is known as "pink Milford," and the entire product of the Connecticut quarry was engaged. For the exterior, 490 000 cu. ft. of cut stone were used, and 60 000 cu. ft. for the interior of the building. This granite was quarried and cut within a period of 18 months, stored at the quarry, and shipped as needed. The setting of the stonework required 13 months, the average rate being 10 000 cu. ft. per week. The ashlar varies from 8 to 12 in. in thickness, and each stone is anchored by two bronze clips. The setting is in "non-staining"

cement. The columns are built up of granite drums, 4 ft. 6 in. in diameter, having an average depth of 6 ft., and weighing from 4 to 6 tons.

Brickwork.—About 15 000 000 common brick were used in the wall construction. In addition, about 1 100 000 cream-colored mottled brick were used for facing the driveways. Gray wire-cut brick were used for the walls of the train-sheds, the west arm of the main waiting-room, and the various parapet walls of the roofs.

Enameled brick were used in the carriage driveway halls, baggage-checking and parcel-rooms, ticket office and cab offices, for facing the retaining walls under 33d Street, and in the elevator shafts throughout the building.

The driveways are paved with a special grade of re-pressed vitrified clinker brick, the total number used being 650 000. The brick was selected, after careful investigation of different pavings, with the view of obtaining a surface which would give proper foothold for horses on the grade and be durable under traffic. This brick was purchased subject to the requirements that the maximum absorption should not exceed 3%, and that the crushing strength should not be less than 4 000 lb. per sq. in. There were two sizes, the larger were 4 by $9\frac{1}{4}$ by 4 in., with square edges, and were used on the level portions of the driveways; the others were $2\frac{1}{2}$ by 7 by $2\frac{3}{8}$ in., with beveled edges, and were used on the inclined portions. The floor was prepared for the brick by laying a 1-in. foundation course of cement mortar to receive the water-proofing, and this was covered with a cement course of the same thickness. Across the width of the driveways concrete ribs were laid at intervals of about 7 ft., over which the water-proofing was extended, and the trays formed by these concrete ribs were filled with sand 3 in. in depth. On this was laid a 6-in. stone-concrete foundation to receive the paving brick, laid in cement mortar. These precautions were adopted to reduce the noise of passing vehicles to a minimum, and to prevent the pavement from moving down the incline.

Interior Cut Stone.—All walls, lintels, and copings are of Bedford stone. The interiors of the arcade and of the main waiting-room are finished partly in travertine, and partly with an artificial cement composition, devised by Mr. Paul Denivelle to produce the effect of real travertine; it is composed of Berkshire white and ordinary cement, white quartz sand, and iron oxides, compounded so as to produce the

FIG. 1.—EIGHTH AVENUE FACADE OF STATION BUILDING.

FIG. 2.—CONCOURSE ROOF CONSTRUCTION.

peculiar lining, voids, and color blends characteristic of the genuine travertine. The blocks were cast in moulds, the slabs being $2\frac{1}{2}$ in. in thickness, and having reinforcing material and anchors for attaching to the walls. The main waiting-room ceiling is of plaster, in deep octagonal coffers, which were cast in moulds on the scaffolding used for the general construction of the waiting-room roof and interior finish. A certain amount of coloring matter added to the plaster makes the tone of the ceiling blend and harmonize with the travertine walls, thus requiring no external decoration. The coffers are 10 ft. in size, are reinforced with steel rods and wire mesh, and are hung from the overhead steel furring by steel anchors.

The exteriors of the ticket offices, parcel-rooms, and booths in the main and sub-waiting-rooms, as well as the trim, partitions, counters, etc., in the dining- and lunch-rooms are of Botticino marble. The public toilet-rooms have trim and partitions of Carrara glass, a milk-white material which is non-absorbent. The trim in the office toilets is of pink Tennessee marble, and this material is used for the wainscots in the hallways.

Fire-Proofing.—A reinforced concrete flat-arch system is used for the floors. The minimum thickness of concrete surrounding the reinforcing rods is four times the diameter of the rods. In general, spans of 6 ft. or more in floors and roofs are of stone concrete in slabs 5 in. thick, and spans of less than 6 ft. are of cinder concrete, with the exception of all tiers below the street level, where stone concrete is used. The reinforcing rods are $4\frac{1}{2}$ in. from center to center, and the ends are turned down over the top flanges of the beams.

Loading.—The following live loads were used in designing the building steel and the floor system:

	Pounds per square foot.
Waiting-room, main and exit concourses....	150
All other floors below street level..........	300
The entire street floor....................	150
Sidewalks	300
Floors in kitchen.........................	150
All other floors above street................	100
Roofs pitching more than 20°.............	30
All other roofs..........................	50

Column Protection.—All columns under and supporting the station in any way are protected by fire-proofing, in accordance with the Regu-

lations of the City Building Department. This fire-proofing consists of a concrete grout filled in between the column and a ½-in. steel-plate jacket which surrounds it for a height of 8 ft. 6 in. above the platform level; above this casing the concrete surface is exposed.

Ornamental Iron.—About 2 500 tons of ornamental ironwork, including cast iron and steel, were used in the interior fittings, especially in the stairways, fences, and in the concourse enclosures for the elevators and lifts.

Vault Lights.—About 155 000 sq. ft. of vault-light construction were used in and around the building. This unusual amount was required to secure light under the sidewalks for the interior passageways and under the concourse floor for the exit concourse and the platforms. The exterior vault lights are of the reinforced-concrete type, with circular glass prisms set with lead rings around each, to prevent shaling. The interior vault lights are of the same type, without rings, except that square instead of round glass was used.

Roofing.—In general the extensive roofing system is of tile covered with No. 16 sheet metal, in part with flat and in part with cleated, or standing, seams. Instead of copper, "Monel metal," an alloy of 60% nickel and 40% copper, was used. This metal is a natural alloy, having approximately the composition of "German silver," and has the well-known non-corrosive properties of that alloy; while strong, it can be readily flanged and soldered, and weathers to an agreeable tone. Of this metal 150 tons were used. The flat roofs are covered with self-glazed tile 1 in. thick. Fig. 2, Plate LXXIV, is a view of the concourse roof construction.

Interior Woodwork.—The furniture in the lunch- and dining-rooms is of Italian walnut; the office trim is of Betula walnut; and the trim in kitchens and service-rooms is of ash. All doors in the public entrances and rooms have long, clear-glass panels; communicating doors between offices have long, raised panels. The doors from corridors to offices are the two-panel type, with transoms above. Base, chair rail, and picture mouldings are used in all offices; and in corridor walls, where borrowed light is required, partition sash have been set.

Flooring.—Maple flooring is used throughout in the offices, and in the Y. M. C. A. section. The general waiting-room and sub-waiting-rooms, lunch- and dining-rooms, Seventh Avenue vestibule, arcade, public lavatories, and barber shop, have marble floors. Cork floors have

been used in the spaces where employees stand, such as the ticket offices, serving pantries, and behind lunch and cigar counters. Granolithic floors are used in the baggage-room, baggage passageway, and kitchens.

Building Erection.—The work of preparation for the building construction began with the sub-surface work in the excavated plot, as turned over to this Department by Mr. Noble. As soon as a sufficient area at the track sub-grade was secured, the work of excavating and constructing the substructures and building foundations was begun. This resulted in the successive completion of the foundations, beginning at the southeast corner of the building, and the erection of the steel frame, from this point working in both directions around the outside of the building, until it was enclosed at the Eighth Avenue side. The masonry followed the steel erection, and the building was finally enclosed at about the time when the entire terminal yard area was excavated to sub-grade. All materials were delivered at the site by trucking through the streets, as railroad connections were at the time impracticable.

The building methods were not unusual, aside from the magnitude of the structures, the amount of erection plant required, and the force engaged. The structural steel was stored and sorted in the plot under the building, and erected by cranes. The granite was delivered around the building to timber staging over the sidewalks and supported from the track level; the erecting was done by derricks set on the inside floors. All other materials were delivered in the building as construction progressed. The general waiting-room, because of its great height (150 ft. from floor to ceiling), required the installation of extensive and massive falsework, which would permit of the convenient and expeditious erection of the steel frame and roof trusses, setting the granite, lining the interior walls, hanging the ceiling, and finishing the interior. This falsework was of 12 by 12-in. vertical posts, at 12-ft. centers, braced with 3 by 12-in. planks, thoroughly bolted, and contained about 500 000 ft. b. m. It was erected in six stories, with runways at each floor. The falsework was equipped with stairways and hoists for loading material on platforms at the different floors. Six stiff-leg derricks, each of 20 tons capacity, having 50-ft. booms, were erected on this staging. Because of danger from fire, the falsework was equipped with an elaborate system of high-pressure fire-protection pipes, with hose outlets at every level, and was constantly patroled by firemen.

Furthermore, it was thoroughly wet down twice a week. Throughout the entire building construction, the temporary system of fire protection and patrol service was maintained, and, although a few minor fires occurred, they were arrested before they had time to spread, no damage to the building resulting in any case.

Constructive Data.—The following is a list of the construction dates, and of quantities of the more important materials used in the building:

Building foundations begun	June 1st, 1906.	
First steel column erected	May 27th, 1907.	
First stone of masonry set	June 15th, 1908.	
Finished exterior masonry	July 31st, 1909.	
Building substantially completed	August 1st, 1910.	
Maximum number of men employed	4 240	
Average number of men employed	1 800	
Granite, exterior	490 000	cu. ft.
Granite, miscellaneous and in concourse	60 000	"
Marble, interior	24 000	"
Travertine	71 580	"
Artificial stone	11 600	sq. yd.
Concrete fire-proofing, cinder	243 000	cu. ft.
Concrete fire-proofing, stone	720 000	"
Granolithic floors	310 000	sq. ft.
Marble floors	85 000	"
Cork floors	11 000	"
Maple floors	147 000	"
Terrazzo floors	10 000	"
Vault lights	155 000	"
Brickwork, all kinds	17 000 000	brick.
Terra cotta furring and partitions	600 000	sq. ft.
Roofing, metal	300 000	"
Roofing, tile	150 000	"
Roofing, skylights	83 000	"
Structural steel	27 000	tons.
Ornamental iron	2 500	"
Glazing	80 000	sq. ft.
Plastering	85 400	"
Painting (area)	2 800 000	"
Cement	64 000	bbl.

Station Facilities.

Of the more important operating features of the building, those involving engineering problems will be described in some detail. They

were planned only after extended Committee and engineering consideration, to utilize to the fullest the available space in the building, with the greatest flexibility of operative methods, and for the future growth of business up to the capacity of the terminal as a whole.

Operating Arrangement.—The station tracks are arranged for both through and stub-end operation, but, for the majority of the movements, it may be considered a stub-end station. Through trains on both the Pennsylvania and Long Island Railroads are handled by electric locomotives; suburban trains of the Long Island Railroad, by multiple-unit motor cars. There are eleven station platforms, serving twenty-one tracks; of the latter, sixteen (Nos. 1 to 16, inclusive, numbering from the south side) are normally assigned to the Pennsylvania service, and all communicate with the North River Tunnels. The remaining five tracks to the north (Nos. 17 to 21, inclusive) are assigned to the Long Island service.

Pennsylvania Railroad trains—except certain short-distance expresses from Philadelphia, and certain locals—after unloading, proceed through the 32d Street and East River Tunnels to a large terminal yard in Long Island City, where they are turned, cleaned, and made up for the return trip. The short-distance expresses are turned and stored in the station yard. All Long Island trains are at present cared for in the station yard; the expresses are switched into the north yard, and locals, after unloading, are tail-switched from the two tracks adjoining the incoming platform (No. 11) to the tracks adjoining the outgoing platform (No. 10). In case of necessity, either platform, No. 10 or No. 11, may be used for either arriving or departing passengers, or platforms Nos. 8 and 9 may be similarly utilized. The 33d Street Tunnels are normally used for Long Island Railroad business exclusively, these tunnels communicating with tracks Nos. 14 to 21, inclusive, but the 32d Street Tunnels may be used in emergencies, thus making all station tracks, except the southernmost four, available for Long Island Railroad service.

Platforms.—The original plan of the station contemplated following the usual American practice of making track platforms about 9 in. high above the rails. Detailed development of the station facilities, however, indicated that low platforms are open to some serious objections, and a departure was decided on in the adoption of the English standard practice, making the platforms flush with the car floor, a

decision which may have a far-reaching effect in the future on the practice in other stations in America. The general features of this platform are shown on the lower part of Plate LXVII. The controlling reasons for the adoption of high platforms in the New York Station were:

(*a*) The greater ease in loading and unloading cars; an advantage which will be appreciated by passengers who are infirm, or who have hand-baggage;

(*b*) The saving in time of loading and unloading trains; this will tend to prevent congestion on narrow platforms, and is an important factor in utilizing to the fullest the station facilities, especially in local excursion and commuter services;

(*c*) A saving of about 4 ft. in the vertical lift between the platforms and the street; this is also an important advantage in a station depressed below the street level;

(*d*) The elimination of the dangerous practice of crossing tracks at grade, a consideration which applies to employees as well as to passengers, and has special force in a station where the view is obstructed by columns, etc.;

(*e*) Incidentally, they permit the convenient use of hydraulic power for operating the elevators and lifts, giving space under the platforms for the machinery and piping; they also provide space for housing the signal and certain other electrical apparatus.

The usual objections cited against high platforms for steam railways had not controlling weight, in the case of this terminal station, because of its location and the type of equipment adopted for other reasons, thus:

(*a*) The new steel cars used for all purposes have vestibules with side-doors arranged so that they can be opened without requiring the trap over the steps to be lifted, and the fascia over the doors is of sufficient height to permit passengers to walk out on the platform level;

(*b*) The difficulty of handling baggage to and from trucks at the cars is minimized by using a special truck, with its platform only 9 in. above the car floor;

(*c*) The arrangement of lifts and cross-trucking subways under the tracks makes it unnecessary to truck across the tracks at grade;

(*d*) The first cost of the high platforms was not excessive, because of the cheapening of other constructive features of the station, such as the lifts, stairways, piping, signals, etc.

There are eleven passenger platforms under the station, varying in width from 20 to 40 ft., and in length from 750 to 1 170 ft. All are "island" platforms, having a track on each side. The total platform length adjacent to passenger tracks is 21 500 ft. In addition, there are "island" platforms west of the Station and under the Post Office which are used exclusively for mail car purposes. Space has also been assigned for two additional platforms at the south side of the yard near Ninth Avenue, for future Express Building purposes. The platforms are of reinforced concrete with edges set 3 ft. 10 in. above the top of the track rails, and 5 ft. 3 in. from the center of the track. These standards allow a normal clearance of 3 in. from the side of the widest car, and place the top edge always somewhat below the car platform, due allowance being made for wear, loading, and variation in equipment.

TABLE 3.—ELEVATORS AND LIFTS.

Elevators, etc.	No.	Size of cars.	Capacity, in pounds.	Capacity, in passengers per hour.	Speed, in feet per minute.	Lift, in feet.
Passenger elevators..	11	5 by 10 ft.	2 500	750	200	17
Baggage lifts.........	21	7 by 15 ft.	7 500	......	100	28
Post Office lifts.......	4	6 by 15 ft.	7 500	......	100	52 to 70
Office elevators.......	10	5 by 5 ft. 10 in.	2 500	......	300	40 to 70
Dumb-waiters........	2	4 by 3 ft.	400	......	200	54
" "	4	2 by 2 ft.	100	......	200	37
Escalator.............	1	Stair 4 ft. wide.	......	9 000	85	25

Water pressure used at hydraulic lifts, 270 lb. per sq. in.

Elevators and Lifts.—The location of the building over the tracks, and the peculiar arrangement of the public spaces and the offices, as well as the great area covered, made the planning of the necessary system of vertical conveyors for passengers and freight a difficult matter. Services were to be provided for:

(1) Passenger elevators from the track platforms;

(2) Baggage lifts between the baggage-room, the platforms, and the subways underneath;

(3) Office elevators from the street level to the floors above;

(4) Service elevators and dumb-waiters between the restaurants and the kitchens;

(5) Moving stairways (or escalators) from the concourse to the street level;

(6) Lifts and conveyors for handling mail from the trucking subways to the main floor of the Post Office.

In general, it was determined that each platform should be provided with a passenger elevator operating between the train level and the exit concourse. It was not possible, because of the plan of the building, to operate these to the street level, nor could they be made of sufficient capacity, without sacrifice of platform space, to handle entire train loads. The arrangement adopted gives one elevator for each platform, eleven in all. Three of these have a lift of 27 ft. 6 in. to the main concourse, and the remaining eight a lift of 17 ft. to the exit concourse only.

Because of the high platforms, baggage trucking cannot be done across the tracks at grade, therefore a very complete system of lifts has been provided between each platform and the baggage-rooms above, as well as to a cross-subway below. Each of the nine long station platforms has been equipped with two lifts, one from the outbound baggage-room on the west side of the building, and one from the inbound baggage-room on the east side. The two short Long Island platforms on the north side of the station have one lift each. The lifts, generally, have a travel of 28 ft. Mail handling to the Post Office, as explained more fully elsewhere, required the installation of four lifts from the platforms to the building proper.

The use of escalators, or moving stairways, from the platforms to the concourse was considered, and, although space was provided for them in the building framing, it was decided to defer their installation until their actual operating necessity was demonstrated. If not needed they would be objectionable, as they would transport passengers to the main instead of the exit concourse; thus they would cause a conflict of passenger movements, and defeat the chief advantage of the double concourse system. The condition at the north side of the station, however, where large commuter travel from the Long Island Railroad is cared for, is special, and at this point an escalator has been provided, leading passengers from the exit concourse under 33d Street to the street level midway between Seventh and Eighth Avenues. By this means passengers are landed without effort in a private street and near the 34th Street cross-town surface car line.

The selection of a suitable operating means for the entire elevator and lift system was made the subject of much study, as many of the conditions were difficult. The great area to be covered (about 20 acres for the building and yard) indicated that electric distribution of power, rather than hydraulic, would be simpler and cheaper. In fact, in the case of low station platforms, as first intended, it seemed to be essential to adopt electric elevators, as there was, in places, no available room for the piping runs and pressure tanks required for a hydraulic system. In case of the baggage lifts, however, it was desired to secure the advantage of the hydraulic plunger type, because of the ease and accuracy of control, simplicity, and absence of machinery and counterweights over the platforms and tracks. Therefore, when high station platforms were adopted, it was found entirely practicable to use hydraulic baggage lifts and concourse passenger elevators, placing all piping and apparatus in the space under the platforms.

The conditions were somewhat different with the office elevators and kitchen dumb-waiters, as their plungers would interfere with clearances over the tracks, and therefore electric elevators were adopted for these services.

Power for the hydraulic lifts is furnished by pumps in the Service Plant, and current for the electric elevators is taken from the general traction power mains through a special switch-board connection in the Service Plant. The electric elevators, being over the platforms and tracks, required special safety precautions to arrest a falling car or counterweight; therefore, all elevator shafts were provided with air-cushion wells, from 8 to 10 ft. deep, and Cruickshank arresters for the counterweights. Thorough tests were made of the efficiency of these devices on all elevators; for instance, a fully loaded car was cut loose at the top of a shaft, allowed to drop freely 70 ft. into the cushion, and was brought to a stop without spilling water or breaking an egg in the car; the air pressure developed in the cushion was about 16 lb. per sq. in.

Gates and Control.—The gate and control systems for the baggage lifts are worthy of mention. To provide against the possibility of a baggage truck breaking through a lift-gate at the platform level, there are collision-proof gates of steel plates and reinforcing angles at all entrances to the lift shafts. These gates are of the disappearing type,

moving in slots in the hatchways below the platform level. They are partly counterweighted, and are operated by compressed air, controlled through levers on the car and from the outside. A gate can be opened only when a car is at rest at the landing; if a car leaves the landing while the gates are open, they will close automatically. Either gate for a car can be operated independently.

In order to obviate the necessity of having an attendant at each lift, a special apparatus was installed to operate the hydraulic starting mechanism through electric control. This consists of push-buttons, located at each landing and on the car itself, electric circuits to a controller-board, and an electrically-operated pilot-valve on the board, controlling the hydraulic valve mechanism proper of the lifts. A car can thus be called for or sent to any landing automatically by pressing the button for that landing. The car cannot be operated by the push-button control if any hoistway gate is open, and when a car is at rest at a landing and the gate is open, the car-operating rope is locked against movement until the gate is closed; furthermore, after a button has been pushed to send, or to call the car, all other buttons are inoperative until the car has moved to, and come to a stop at, the landing called. If a gate is opened while a car is moving, the car will stop at the next landing toward which it is moving. In addition to this automatic control, the lifts may be operated in the usual way by hand-ropes.

Baggage Handling.—The baggage facilities have been planned for as rapid service as consistent with the very large platform area served, and to avoid long-distance trucking on the platforms, which are necessarily somewhat narrow and obstructed by building columns. Therefore, two baggage-rooms have been provided; one at the east side, adjoining the main waiting-room and the driveways, for delivering to and from wagons, for checking and for arriving baggage from trains; and one at the west side, adjoining Eighth Avenue, for delivery to trains. The east room contains all the usual facilities for weighing, checking, storage, and offices for the operating force, and for the Transfer Company, and is the one to which the public has access. The west room is chiefly a passageway for reaching the various platform lifts. The two rooms have communication by a trucking passageway under 31st Street and under the Seventh Avenue front of the building. The following data relate to baggage handling, in terms

of amount handled at present in summer; the ultimate capacity of the facilities provided is much greater, of course, than the figures given:

Estimated quantity of baggage handled per hour.	220 pieces.
Total quantity per 24 hours..................	6 025 "
Storage capacity	6 000 "
Length of run from east to west room.........	1 350 ft.
Time required to deliver truck load to car at west end of platform.....................	8 min.
Number of baggage scales....................	4
Capacity, total	20 000 lb.
Capacity, weighing	10 000 "

Baggage Trucks.—There are four kinds of baggage trucks for station uses. The special trucks, used for general baggage purposes, were designed and built by the Motive Power Department, at Altoona, and are similar to those in use elsewhere on the road, except that they have drop frames, with floors flush with the track platforms, for greater convenience in loading from the cars. These special trucks are automobile and trailing, both of the same construction, except that the former are equipped with electric motors and storage batteries. The battery is of twelve cells of 200 ampere-hours capacity, with a maximum discharge rate of 50 amperes. The batteries are recharged at a stand in the 31st Street baggage passageway, where hand-cranes, charging racks, and electrical connections, communicating with a switch-board in the Service Plant, have been placed. The charging current is 25 volts, and is supplied by small motor-generators in the above plant, as a part of the auxiliary power system. Other data regarding these trucks are as follows:

Size of platforms..........................	3 ft. 8 in. by 12 ft.
Height above floor....................	$9\frac{1}{2}$ in.
Speed, in miles per hour.............	6
Shortest turn	15 ft. radius.
Weight of truck, light................	2 600 lb.
Capacity of truck.....................	4 000 "

The following trucks are provided for different station purposes:

Electric baggage trucks.........................	25
Trailer baggage trucks of the same type, without motors	25
Mail-handling trucks, without motors.............	25
Funeral trucks	6

Train Indicators.—In the main concourse, at the head of the stairs to each track platform, there are gates and illuminated signs describing the destinations and the departure times of trains. There are forty-four of these indicators. They consist of cast-iron columns, 16 ft. high, two at each gate. The top of each column is four-sided, and contains mechanism for moving steel tapes opposite the openings on the four sides simultaneously; on these tapes the numerals from 1 to 10 are enameled. The mechanism is operated by a crank key through vertical rods and gearing from sockets near the bottom of the post; by turning this key the tapes may be set to indicate any desired departure time. The posts also carry a four-sided card box, in which destination signs are displayed; these are placed in a frame, operating in guides in the post, and raised into position by a hoisting-drum mechanism.

Train-Starting System.—As the distance between the gates on the main concourse and the trains at corresponding station platforms is considerable, it was necessary to have a system for quick communication between the gateman, the conductor of the outgoing train, and the train director in the signal cabin, in order to insure prompt control of the starting of trains at the scheduled time.

At the head of the stair leading to the platform there is a push-button and lamp indicator; and on each platform at four different points there is inserted in the column an instrument containing a switch, a push-button, an indicating light, and, higher on the same column, another light. There is an instrument for the same function in the interlocking cabin controlling the train movement out of the station. Fig. 1, Plate LXXV, is a view of a column at the platform level, showing the train-starting device and telephone set. All these various devices are interconnected by electric wiring, and operate as follows: About 1 min. before the train is to leave, the conductor inserts a key in the train director's instrument, thus showing the number of the train for which the announcement is given. The director then moves a lever, which closes the circuit and lights a lamp in the conductor's box and at the platform gate to indicate to both conductor and gateman that the route has been set for the departure of the train. When a gateman closes his gate at the train leaving time, he pushes a button, extinguishing the indicating light and, at the same time, by the lighting of the lamp at the top of the

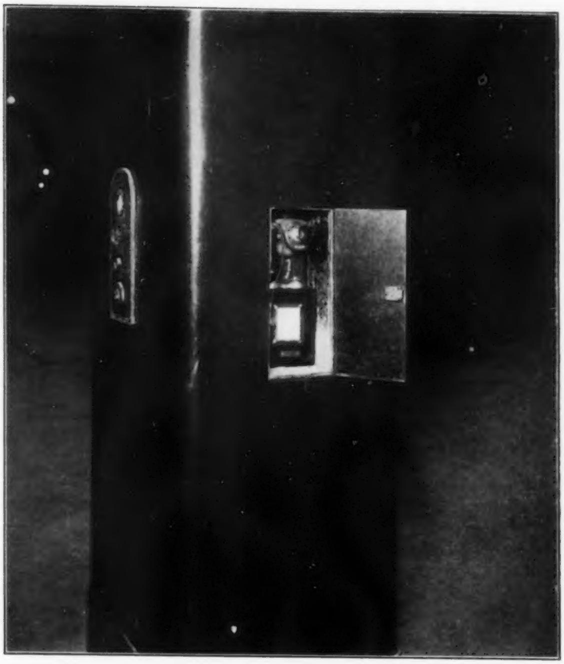


FIG. 1.—COLUMN AT PLATFORM LEVEL, SHOWING TRAIN-STARTING DEVICE AND TELEPHONE SET.

FIG. 2.—PIPING AND HOT-AIR DUCTS IN STATION BASEMENT.

column, notifying the conductor that the gate is closed. When the passengers are aboard the train the conductor operates a push-button circuit-breaker, extinguishing all lights and restoring the apparatus to normal.

Clock System.—Electrically-operated and centrally-synchronized clocks have been provided throughout the station and yard; they are distributed as follows: Four clocks, 6 ft. in diameter, on the exterior façades of the building; fourteen, with diameters varying from 18 in. to 15 ft., in the public rooms; twenty-six in various offices, yard buildings, and signal cabins, and one master-clock, in the train despatcher's office, synchronizing all other clocks.

The clocks are of three different types: motor-driven, impulse-driven, and primary or self-contained. The master-clock is equipped with a transmitter and circuit-closing device to transmit operating and synchronizing impulses to the other clocks, and besides being a close time-keeper, is corrected automatically once in each 24 hours from a signal sent out by the United States Observatory at Washington.

The motor-driven clock is the 15-ft. dial instrument in the main waiting-room; its hands move forward slowly each ½ min. The impulse clocks, comprising those in the public rooms, have a jump movement forward each ½ min., by impulses sent out from the master-clock through the medium of a transmitter. The "primary" clocks in the offices have self-contained winding and operating mechanism, with a winding battery within the clock and arranged to be corrected hourly by impulses received from the master-clock. The primary clocks are synchronized in multiple, and the impulse clocks are operated in series.

A central storage battery of twelve cells, in duplicate, provides the necessary source of energy for operating the entire plant. Each battery is sufficient for one week's work, the re-charging being done alternately from a motor generator in the Service Plant. The total current consumption is about 3 ampere-hours per day.

Nearly all the clocks are lighted by reflection from the general illumination of the rooms, but those on the exterior of the building and in the main waiting-room are lighted by electric lamps in the clock cases. The casings of the clocks on the exterior of the building and in the public rooms were designed by the architects, and at each location harmonize with the general finish of the building.

Pneumatic Tubes.—For the prompt dispatch of messages and small packages, between the different buildings throughout the yard and the offices in the station building, a pneumatic-tube carrier system has been installed. It operates at an air pressure of from 2 to 4 lb. per sq. in., the air supply being taken from the compressors in the Service Plant through reducing valves into low-pressure storage tanks, located at various points throughout the yard and building, and thence piped to the tube terminals.

The tubes are of brass, specially drawn, are of various sizes, and have long-radius turns. Each run is of a single tube, having terminals at the ends to serve for either dispatching or receiving the carrier. The terminals are normally open, and, when the carrier is placed in the tube, the door at that end is closed and air automatically admitted. The door remains closed until the carrier reaches the opposite end of the line; there it trips a trigger which opens an electric circuit and de-energizes the electro-magnet which holds the door shut at the sending end.

The carriers are of leather, and of the required sizes to hold messages, baggage checks, packages of tickets, etc., depending on the service. The length of the tube runs varies from 150 to 1 000 ft., the total length of the system aggregating 7 000 ft.

The following are the lines of intercommunication established:

From the telegraph office on the second floor of the Eighth Avenue building, $2\frac{1}{4}$-in. lines to each of the following:
- Two Pullman offices in general waiting hall.
- Baggage Agent's office in baggage-room.
- Station Master's office at northwest corner of concourse.
- Assistant Yard Master's office in yard under Post Office.

From the two telegraph offices in the general waiting hall, $2\frac{1}{2}$-in. lines to:
- Each Pullman office in general waiting hall.

From Assistant Yard Master's office in yard, $2\frac{1}{4}$-in. line to:
- Signal Cabin "A" in the yard.

From Signal Cabin "A," $2\frac{1}{4}$-in. line to:
- Car Inspector's office in yard.

From the ticket stock-room, 5-in. lines to:
- Two general ticket offices in general waiting hall.

From Assistant Baggage Master's booth in baggage-room, 4-in. line to:
- Two baggage-checking desks in baggage-room.

Lighting.—The engineering considerations leading to the selection of the type of lighting for the Station are discussed in connection with the design of the Service Plant. It is there stated that small lighting units, rather than powerful arc lamps, were adopted. This decision was reached, not only because of the good architectural effect and the agreeable quality of small lights, but because of the effective diffusion secured from numerous sources of light. In general, the treatment adopted for different spaces is as follows:

The street lighting is by lanterns of moderate power, set on posts 16 ft. high, and 45 ft. apart around the building; this system is used extensively in Europe, especially in Paris. The arcade is lighted by side brackets containing clusters of Nernst lamps. The public rooms, except the main waiting-room, are lighted by ceiling chandeliers, consisting of rings of Nernst lamps; the main waiting-room, because of its great height, is lighted near the floor only by two rows of cluster lamps on posts, with a limited amount of side-bracket lighting for wall illumination; the ceiling of this room is intended to be left in semi-obscurity, thus increasing the effect of height at night. The concourse, having its roof and floor largely of glass and ironwork, was a difficult space to light agreeably and effectively; after experiments with various kinds of lights, it was decided that rings of Nernst lamps around the columns, and ring chandeliers for the central spaces, gave the best effect.

For general illumination, the offices are equipped with ceiling lamps; local desk circuits, however, are provided in the base-boards of each room. The platforms are lighted by Nernst units about 20 ft. apart and as high above the floor as local conditions permit. The intensity of the platform lighting, as will be noted from the list, is relatively low compared with other spaces, but is ample.

The question of the best type of lamp for lighting the Station was given much consideration. At the time when decision was necessary, choice lay between the ordinary carbon filament, the metalized carbon filament, and the Nernst. This latter form uses a "glower," made of a mixture of rare metallic oxides, and is not sealed in an exhausted bulb, as are ordinary incandescents. The Nernst lamp gives light of agreeable quality, and its construction is such that the light is radiated from the lower hemisphere without the use of reflectors, thus readily adapting it to general illumination from overhead sources; furthermore, it was decided to be more economical than other forms available.

For these reasons it was decided to adopt it for all general illumination of the Station; in certain special cases, however, such as for the lamp-post clusters in the main waiting-room, for desks in the offices, etc., incandescent lamps, either carbon or tungsten, are used.

Electric current, used in the Station for lighting and motors, is distributed at the required voltages from a main switch-board in the Service Plant through cables to sub-switch-boards, located at centers of the main divisions of the Station; from these boards it is further distributed to local boards in the smaller sections, or in individual rooms, and finally to the room switches in the offices, or to large groups of lights in the public rooms. All the larger public rooms are supplied with current from duplicate sets of feeders, one set of which is connected to an emergency bus in the Service Plant. The main feeders from the Service Plant are carried through the pipe subways under the tracks and in special pipe and wire shafts built in the Station Building walls. The branch circuits are generally carried on top of the concrete floor arches and built into partitions. All feeders and sub-feeders are three-phase; branch circuits are single-phase. The lighting of public rooms is arranged so that half the lights are supplied by each of two feeders.

Miscellaneous power distribution is arranged in a similar manner, the power to the heating and ventilating motors throughout the building being distributed directly to the sub-boards in these rooms from the Service Plant switch-board.

Further data are given below:

Total number of lighting fixtures in Station....	21 000
Total equivalent combined candle-power of lights in Station	335 000
Feet of conduit used..........................	310 000
Feet of wire (No. 14 to 600 000 cir. mils.)........	690 000
Candle-power (mean hemispherical) per square foot, main waiting-room..................	0.45
Other public rooms..........................	0.60
Offices (general lighting).....................	1.25
Main concourse	0.35
Platforms	0.25

Heating and Ventilating.—The heating and ventilating is one of the most important as well as one of the most complicated of the service requirements, and covers the heating of the main station as well

as of the numerous buildings in the terminal yard, and the heating supply for cars standing in the yard, over an area of about 28 acres. The type of heating had a bearing, not only on the kind and quantity of apparatus required, but on the design and construction of the Station Building itself, and therefore the general heating scheme had to be determined prior to the construction of the building, and with an intimate acquaintance of the plans, in order that its service could be made to harmonize therewith.

The problem involved primarily the heating of a building having very large cubical contents, but especially one covering an unusually large ground area, and the fact that the building has no basement or cellar proper introduced special difficulties in installing large heating mains and radiating apparatus. The rooms in the building vary in size from the main waiting-room, 110 by 150 ft. and 150 ft. high, down to the usual dimensions of offices; and the occupancy covers the composite requirements of a railway station, a restaurant, and an office building. Many of the rooms are designed to house large numbers of people, some of the rooms have only indirect communication by windows with the outside atmosphere, and many are below the street level; therefore, it appeared desirable to use a forced-draft heating system, so that the air might be taken from suitable places and that proper ventilation could be had in summer as well as in winter, provided it could be installed and operated at a moderate cost as compared with other practicable methods.

To determine the best system of heating, all things considered, as applied to the local conditions, elaborate calculations and preliminary plans were made for various systems, as follows:

(*a*) Direct radiation in the various rooms, without forced ventilation;
(*b*) Indirect heating from pipe stacks located at central points, the warmed air being distributed through ducts by forced draft;
(*c*) A combination of part direct and part indirect heating.

It appeared that portions of the Station, such as the waiting-rooms, kitchens, restaurants, and toilets, where forced heating and ventilation were desirable, if not actually necessary, comprised at least two-thirds of the total area to be dealt with; and that if the indirect system should be used for places where it is essential, it could be extended to embrace the smaller spaces, such as the offices, without introducing

prohibitive complication or without materially increasing the first cost of installation over that of the combined direct-indirect system. The conclusion was reached, therefore, that the indirect system with forced ventilation, both by draft and suction, should be adopted for all spaces except special isolated places, such as the baggage-rooms, which cannot be entirely closed by doors, and in cases of the small isolated buildings in the yard, where the direct system of radiation from pipes in the rooms should be used.

The next consideration was that of the medium of supplying the heat from a central point, the source of heat being the boilers in the 31st Street Service Plant. It was concluded that low-pressure steam, either live or exhaust, conducted to the building through pipes was impracticable, on account of the great area to be served, the very large dimensions of the pipe mains, pipe expansion troubles, and the lack of opportunity for draining properly the complicated system of return piping. Furthermore, exhaust steam thus used would cause excessive back pressure on the engines; live steam at high pressure would reduce the diameter of some of the pipes, but would still leave unsettled the question of proper drainage, and would make it difficult to control the pressure at widely separated points. The cost of operation would also be high, as no advantage could be taken of the economy to be gained by passing the steam first through the engines to produce light or power. The remaining method, namely, the use of water heated at a central point near the engines by exhaust steam from them, and distributed by pumping through a piping system to locally placed stacks in the building, appeared to be best. Such a system involves small piping only, is free from drainage troubles, is convenient for regulation, and gives the best quality of heat under varying weather conditions.

While, as above indicated, it was concluded that the water system of distributing heat was the only one filling the practical necessities of this particular case, an estimate was made of the comparative first cost of all systems, and it was found that the three available did not differ greatly in this respect, and that the indirect system with hot water could be installed at as low first cost as that of any other.

Comparison was made of the operating cost of both the direct and indirect systems, on the basis of both live and exhaust steam-heating means. It was found that the lowest operating cost would be obtained by using exhaust steam to heat water circulation with direct-heating

radiators in the rooms. Forced ventilation, however, which as above stated was considered a necessity, would not entail a greatly increased operating cost by the use of indirect-heating stacks, as the supply of exhaust steam would be ample for all except the severest weather.

Steam from the boilers in the Service Plant is passed through the various engines used for lighting, for air compressors, pumps, etc., and the exhaust is taken into tubular water heaters. Motor-driven centrifugal pumps circulate water through these heaters by the closed-pipe system into and through nine heating chambers in various parts of the Station Building. From these the water returns to the circulating tanks and is used again. Connections are also made to the water heaters, so that live steam can be used when necessary. Through the heating stacks in the building fresh air is delivered from hoods on the roof, being drawn by fans and forced through a system of galvanized sheet-metal ducts into the various rooms of the building. In general, the warm air is admitted at or near the floor line and the foul air is drawn out by suction fans at or near the ceilings of the rooms. The design of the building is such that the heating stacks may be located in places where they occupy little valuable space, and the heating ducts are in most cases run in the ceilings of the rooms and passageways. Fig. 2, Plate LXXV, shows a typical portion of the piping and hot air duct system in the Station.

The total volume to be heated in the building is 10 280 000 cu. ft., and it was estimated that to provide proper heating and ventilation under maximum conditions would require the circulation of 2 000 000 lb. of water per hour at a temperature of 200° Fahr. with a return temperature of 160 degrees. The fans (Fig. 1, Plate LXXVI) and local stacks (Fig. 2, Plate LXXVI) used for transferring this heat have a capacity of 37 000 000 cu. ft. of air per hour raised from zero to 130° Fahr., requiring about 77 000 000 thermal units.

The total loss for exposure, with the outside temperature at zero and the inside temperature at 70° Fahr., is about 30 000 000 B. t. u. per hour, and the air entering the rooms at such times is heated to about 120° Fahr., with a discharge velocity of about 300 ft. per min.; an average drop of 10° is allowed for losses in the air ducts.

The heating surface in the nine heating chambers aggregates 76 500 sq. ft., made up of cast-iron cellular units in fifteen different stacks. Each of these stacks is provided with a motor-driven fan,

the motor being belted to the fan pulleys, so that the fans may be driven either by single motors or in groups. The fans are multi-vane, and the motors are of the three-phase induction type, varying in horse-power from 20 to 40. The fan capacities vary from 15 000 to 75 000 cu. ft. of air per min. Screens are provided in the fan chambers for cleaning the air.

Ventilating System.—Galvanized-iron ducts and exhaust fans are provided for ventilating purposes in addition to the heating fans. There are twenty-one different fans for the purpose, varying in capacity from 4 300 to 32 000 cu. ft. of air per min., with belted motors varying in capacity from 2 to 10 h.p. The total capacity of these ventilating fans is 43 000 000 cu. ft. of air per hour, or sufficient to change the air in the different sections of the building from three to ten times per hour, depending on the occupancy of the particular space.

As the rooms used for different purposes communicate with one another, it was necessary, in designing this heating and ventilating system, to provide for suitable differential pressures in the rooms, in order that odors should not be communicated from one room to that adjoining; this is especially necessary in the ventilation of the kitchen and serving-rooms and of the various toilet-rooms. The pressure in these rooms, therefore, is maintained below that of those adjoining, so that the ventilation is into them rather than the reverse. The ventilating system of flues and fans, therefore, is divided into two sections, giving entirely separate ventilation for the kitchens and toilet-rooms. Ventilation of the toilet fixtures is accomplished with a local vent in a closed space back of each fixture, which in turn is connected to an exhaust vent, so that, in operation, a current of air is continually drawn into the toilet-rooms, passed through the fixtures, and discharged above the roof of the building.

In order to economize in the amount of heat required in the large public rooms in very cold weather, arrangements have been made to by-pass the discharge from the ventilating fans, either in part or entirely to the outside atmosphere, or to the heating stack chambers; thus the warmed air from the rooms may be used again in any proportion desired. The number of separate register openings required throughout the building to control the inlet and egress of air for heating and ventilating is 3 000.

Plumbing.—The plumbing comprises the extensive system of piping

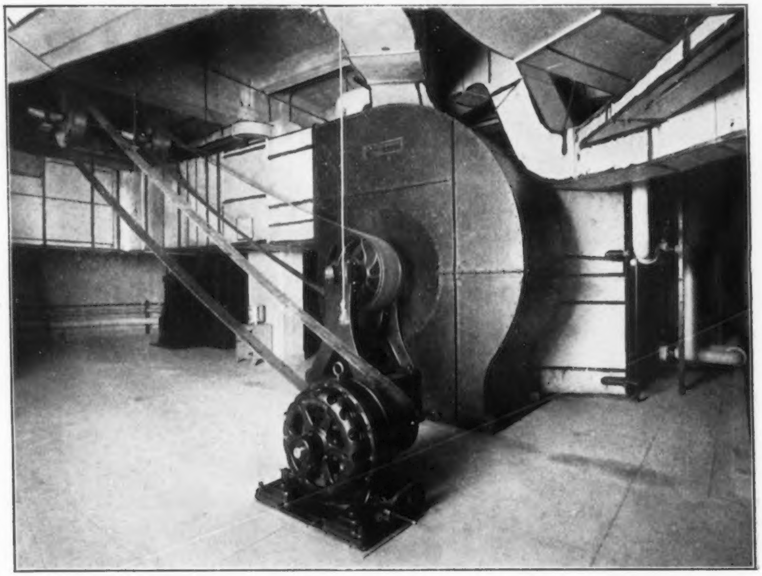


FIG. 1.—HOT-AIR FAN-ROOM IN STATION.

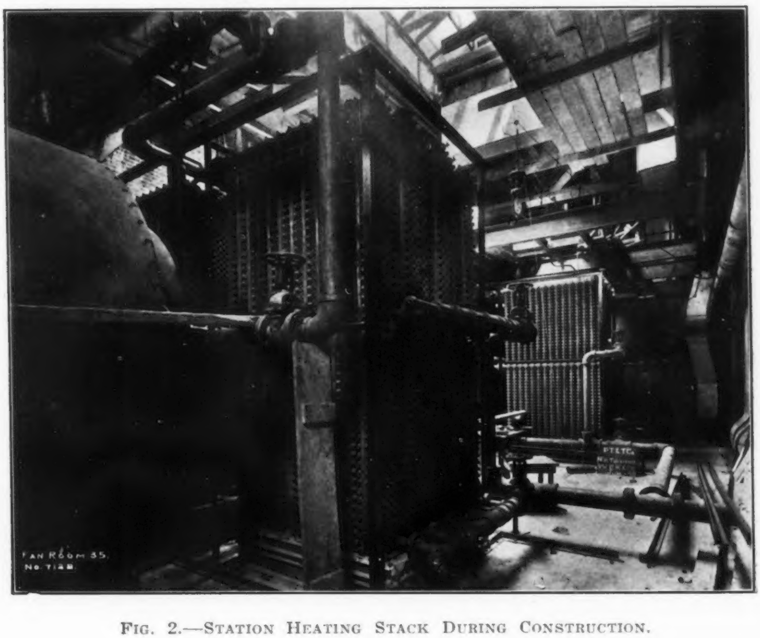


FIG. 2.—STATION HEATING STACK DURING CONSTRUCTION.

and apparatus for toilet-rooms and lavatories throughout the Station Building and yard. In the various terminal buildings, exclusive of those in the Post Office, 58 separate toilets have been installed. About one-third of these discharge directly into the sewers, and the remaining ones through ejectors. The capacity of the various toilets was determined by careful study of established practice for office buildings and public places. In the rooms of the Station alone, 846 fixtures have been provided, and a total of 932 throughout the entire area.

The main public toilets are noticeable for their size and completeness of arrangement. They are located immediately under the sub-waiting-rooms, on the level of the exit concourse, and are accessible by lobbies from the sub-waiting-rooms, and directly from the exit concourse. The men's toilet consists of a free room containing 51 closets, 71 urinals, and 35 wash-basins, and a pay toilet with 29 closets and basins in separate enclosures. The women's toilet is similarly arranged, and contains 100 free closets and 33 wash-basins, and a pay toilet of 44 compartments having toilets and lavatories. The interior of these rooms is finished with marble floors and Carrara glass side-walls and partitions between fixtures.

All piping has been placed in galleries or compartments behind the fixtures, and all pipes are exposed for inspection and repair. This arrangement is somewhat unusual because of the limited vertical height between the floors of the waiting-rooms and the clearance over the track level, requiring that the fixtures have side outlets.

Water supply for the toilets is obtained from the pumps in the Service Plant; the flushing water is the waste from the jackets of the compressors and refrigerating plant. More than 51 miles of piping were used in the runs to the various plumbing fixtures.

The toilet-rooms which have their waste at a level below that of the city sewers in the adjoining streets and avenues required a special system for elevating and discharging the waste to the level of the sewers. The ejectors designed for this purpose use air at a pressure of from 25 to 30 lb. per sq. in., operating automatically. They are located at twenty-two central points in niches in the subways under the tracks. Fourteen of the ejectors have a capacity of 50 gal. and eight of 100 gal. per min. Separate lines of pipe supply air from the compressors in the Service Plant to storage tanks near each group of ejectors; at these points reducing valves are placed to give the 30-lb.

pressure required. Pipes from the ejectors are run through the subways to the nearest convenient points adjacent to columns of the building and viaducts, up which they are run to the street connections; the lift is generally from 40 to 70 ft.

Cooled Drinking Water.—Pure drinking water of uniform temperature is distributed throughout the building by a separate system of piping. There are 158 special drinking fountains in the public rooms, the restaurant, and the office corridors. Water from the city mains is delivered to the Service Plant in 31st Street, where it is filtered, cooled to 40° Fahr., and pumped to the fountains. In the public rooms the fountains are supplied with vending machines which dispense paraffined-paper drinking cups at 1 cent each.

Pipe Gallery.—In connection with the description of the various piping systems, reference has been made to the difficulty of finding proper space for the pipes and heating ducts in a building which has no cellar or basement; many of these pipes are of large size, and should be located where they can be inspected from time to time. The problem was finally solved by constructing in the floor truss system, under the main waiting-rooms and the baggage-rooms, a pipe gallery having an extent of about 2 acres. This space is T-shaped, and of a height varying from 5 to 7 ft.; it is intersected by numerous trusses and lattice bracing, but is sufficiently open to permit of installing a large part of all the piping needed for the building services above, as well as heating ducts for the public rooms above. It communicates directly with the pipe gallery under 31st Street leading to the Service Plant. From this gallery a number of the vertical pipe shafts, leading up into the building, are reached, so that the main runs of pipe are everywhere accessible. All steelwork in the gallery is fire-proofed by encasing in cement, according to the Regulations of the City Building Department.

Fire Protection.—The fire protection of the building is described in connection with the general system under the heading, "Station Yard."

Watchmen's Registers.—Watchmen patrol the building hourly, and each carries a time register consisting of a "Newman" portable registering clock. He is required to record the time at stations by inserting in the clock a key kept in a special box at each station. There are key boxes at thirty-eight points in the building, and they are placed

so as to require the watchman to pass through all important sections in making a round.

Restaurant.—The dining- and lunch-rooms each have a serving-room attached, with a kitchen, store-rooms, refrigerator-rooms, offices, and help quarters above. All these are thoroughly appointed with most modern apparatus throughout. The seating capacity of the dining-room is 500 persons, at 125 tables; of the lunch-room, 40 tables, or 160 persons, and 93 stools at the counters.

The refrigerator contract called for the installation of forty cold-storage rooms, the largest of which is 34 by 42 ft., containing about 12 000 cu. ft. The insulation is of compressed sheet-cork of the best grade; the inside walls are of Carrara glass, and the floors are of tile. The boxes are cooled by overhead brine pipes, receiving circulation from the refrigerating machines in the Service Plant. The temperatures in these boxes vary from 8 to 38° Fahr., according to the purpose served.

The kitchen contains twelve roasting ovens, three charcoal broilers, and three gas broilers, all erected under a hood 54 ft. long. The pastry-room contains a gas range with ten ovens. The miscellaneous equipment includes electrical apparatus for meat-chopper, potato-parers, knife-cleaners, etc., and the service-rooms have electrically-operated dish-washing machines, steam-tables, etc., etc.

Offices.—The Eighth Avenue front of the building, and eastward on the side streets to the concourse, contains the office section, on three floors. The first floor is devoted to the Depot Master and train staff, with locker-rooms, a hospital, a police department, and funeral-rooms. The second floor contains the offices of the Division Superintendent and staff; the third floor contains the General Offices of the Long Island Railroad, and of the President and staff. On the Seventh Avenue side there are two floors of offices, at present not fully assigned.

Employees' Conveniences.—In addition to the above, the fourth floor, on Eighth Avenue and Thirty-first Street, has been fitted up for the housing and recreation of employees. The Thirty-first Street side contains sleeping-rooms, toilet and bathing facilities; the present capacity is 175 beds. The entire Eighth Avenue front has been fitted up for a Young Men's Christian Association, with an assembly hall, lecture-rooms, library and reading-room, billiard-room, bowling-alley,

and gymnasium; there is also a large lavatory with shower-baths, and a locker-room. All the above have been completely furnished.

Various small buildings, listed elsewhere, have been provided at the track level, under the Station and in the yard, for toilets, locker- and waiting-rooms, for employees on and off duty, or awaiting trains.

Post Office Building.

The change in destination of all through trains of the Pennsylvania system from the Jersey City to the up-town New York terminal required a revision in existing facilities for handling mails, of which about 250 tons are cared for daily. Anticipating the growing importance of an up-town central distributing Post Office, the United States Government purchased from the Railroad Company a plot of about 400 by 400 ft., immediately west of Eighth Avenue, comprising a portion of the Station yard, as a site for a suitable building. In the deed, the Railroad Company reserved the right of easement for their uses of all space below a plane substantially 20 feet above the yard track level. In conveying the property to the Government, certain necessary agreements were entered into regarding the character of the building, the location of the supporting columns and foundations, and provision for open spaces around the building to admit of light to the tracks underneath. Pursuant to this deed, the Government has planned and is erecting an extensive and monumental building, and, while the Railroad Company is not responsible for the design of the building proper, the necessities of the yard construction work required that the foundations should be put in prior to the time when the building contracts were ready to be let, and simultaneously with the great variety of substructure work in the yard. It was arranged, therefore, that the Railroad Company, in conference with the Architects of the building, should plan and construct these foundations for the Government.

Design.—The following description of the architectural motif of this fine building has been furnished by Messrs. McKim, Mead and White, the Architects, and will be of interest in connection with the history of the general terminal facilities.

"The architects have endeavored, while keeping in mind the practical uses of the building, to give it the monumental character which a Government building of such importance should possess. In general, it may be said that as any building of modern height in New

York is likely to be of an inferior height to those eventually around it, the chance to compete with them will depend largely upon the great scale and unity of its design. For this reason the exterior was planned with as few breaks as possible in its façades, and a columnar motif running through two stories was adopted.

"In determining the character of the building, due consideration was given to the proximity of the Pennsylvania Station, and, in order that it might be in harmony with that building, the style adopted was Roman, and at the same time it was considered that the building must have a quality of its own which would associate it distinctly with the Governmental class of building. Rarely has so favorable an opportunity been presented for producing, on a scale commensurate with the Fora of Rome, such a development of colonnaded and pilastered façades complementary to each other and lending themselves to that unity of scale and style productive of the greatest effect.

"As a matter of composition, the Railroad Station having three pavilions, a building opposite to it, and shorter, with the same number of pavilions, would be an impossibility, and therefore the design was studied with the view to avoid a conflict of central motives; and for this, among other reasons, the design of the Eighth Avenue façade of the Post Office shows two pavilions joined by an unbroken row of columns of the same diameter as those of the Station, an arrangement which brings the two buildings into proper relation, produces the greatest possible effect in the given space, and which is, moreover, an expression of its plan.

"The principal approach to the building on the Eighth Avenue front is by granite steps the entire length of the colonnade, and subordinate approaches are also arranged for by means of granite steps giving access to the north and south fronts of the corner pavilions.

"The façades on the streets are continuous and of the same general motif as the Eighth Avenue front, pilasters being used instead of columns. The street façades are also terminated in pavilions at the westerly ends. The architectural style selected is Roman-Corinthian, that of the Station being Roman-Doric. The exterior material is granite.

"On the ground floor of the building the space allotted to the public is in the form of a wide corridor extending back of and on a line with the colonnade above referred to, back of which corridor is the general working space for distribution, handling of mails, etc., and on the westerly front of the building is a private driveway, connecting 31st and 33d Streets, by which access to the building is given to mail wagons and other city deliveries.

"Directly over the corridor on the Eighth Avenue front on the second-floor level, and in the center of the façade, is located the Postmaster's suite of rooms, and on either side and extending somewhat

back on the north and south fronts are located the rooms of his principal assistants.

"A mezzanine floor, between the first and second floors, and which is wholly back of the public corridor above referred to, is assigned chiefly to swing-rooms, locker-rooms, toilet facilities in connection with both of above, and for record- and document-rooms.

"The Eighth Avenue front of the third story of the building is assigned to the use of the Executive Staff of the Railway Mail Service; other parts of the second, third, and attic stories of the building are assigned generally to the working force of the various Departments.

"The westerly and southerly portions of the basement of the building are assigned to the working force of the Railway Mail Service, and direct communication between this service and the Railroad Company's tracks and platforms, and between this service and the working space for mails on the floor above, is had by means of mail conveyors, elevators, chutes, and other mechanical installations. Other portions of the basement are assigned to newspaper and parcel distribution, mail sack storage and repairs, cashier's stock, etc."

The work for which the Railroad Company is responsible in connection with the Post Office, is the location of the building columns in relation to the tracks and platforms, the planning and constructing of the roofs for the light areas between the building and the street viaducts, and the provision for handling mail between the delivery floors in the building and the cars beneath.

Building Columns.—The columns have been laid out on substantially the same plan and spacing as adopted for the supports of the Station, for a building of approximately the same general height and loading. There are 200 columns in the platforms and between the tracks. The foundations for these columns are of concrete, to the level of the capstones or grillage, constructed by the Railroad Company as a part of its substructure work.

Connecting Roofs.—The connecting roofs consist of a permanent vault-light covering for the areas between three sides of the building and the adjoining street viaducts. The construction takes the form of a generally flat roof of I-beams carrying glass vault lights set in concrete. They are of the usual sidewalk-light type, and give about 40% of the total for glass area. The roofs are set at an elevation of about 6 ft. below the sidewalk level, and are designed to sustain a load of 100 lb. per sq. ft. The roofs have sufficient slope to drain to points 50 ft. apart, from which the drainage is piped to the yard system.

Mail-Handling Methods.—At present about 40% of the entire amount of mail originating in New York for outside points is received and dispatched by the Pennsylvania Railroad. On the heaviest days from 220 to 260 tons are carried in from 12 000 to 16 000 pouches, each weighing about 200 lb. The great bulk of this mail, about 80%, has been heretofore delivered at Jersey City by wagons directly into postal or mail storage cars on side-tracks adjoining the station and convenient for the teaming yard; these cars frequently lie on the tracks several hours while loading and unloading.

Practically all this mail, in addition to that from the Long Island Railroad, and, at a later date, possibly, from the New Haven Road, must be provided for at the new Station, under physical conditions radically different from those existing at Jersey City. The new Post Office Building being over the railroad yard, it is obvious that unlimited track space for standing mail cars, or platform space for loading mail into the cars, cannot be provided; in fact, the value of each foot of space in the yard is so great that provision must be made to handle the mail in the most expeditious manner, and on very limited track room.

The track space assigneed especially for standing mail cars is centrally located in the yard, partly under and partly west of the Post Office Building, and consists of six tracks, adjoining four of the central platforms, having a total maximum storage capacity of twenty-six postal cars. Numerous plans were considered by the Joint Committees of Postal and Railway officials for the utilization of the available track and platform space, in order to load and unload the cars in the shortest possible time and with least manual effort, and it was thought essential to develop a complete system of mail-handling machinery, consisting of chutes and horizontal conveyors for outgoing mail, and horizontal conveyors and bucket lifts for incoming mail, together with vertical lifts for mail on trucks. This system is designed to reach, not only the Post Office Building, but, through the trucking subways under the tracks, any part of the main Station Building, and provides for the mechanical handling of all bulk mail for railway post office or storage cars, and the convenient trucking and elevating of sacks to and from combination cars standing at any station platform. In general, the system may be described as follows:

Incoming Mail.—Mail in less than car-load lots, or in combination cars, will be unloaded as the train stands in the Station, on trucks, and conveyed to the Post Office either along the platforms or, by utilizing the nearest baggage lift, descending to the trucking subway under the tracks, and thence to the Post Office lifts. In the case of car-load lots and bulk mail, the cars will be switched to the tracks adjoining platform No. 4, where the mail will be unloaded manually and dumped into hoppers on the platform. From these hoppers the mail pouches will be pushed automatically by compressed-air rams on a belt conveyor located under the platforms; this belt will convey the bags to a point where a tilting tray operates to transfer them automatically to a vertical bucket lift, which will elevate them into the Post Office mezzanine floor, from which they will be delivered through spiral chutes, and, at the option of the employees, either to the receiving mail platform on the first floor or to the basement of the building.

The compressed-air rams, for charging the pouches from the chutes to the belt conveyors, consist of a pair of tandem cylinders and pistons of different diameters fastened to one piston rod, for the purpose of getting a differential effect; reciprocating action is secured by different pressures obtained through two lines of piping, one supplying and maintaining a constant pressure between the two pistons and the other supplying a variable pressure applied to the outer end of the larger piston. The necessary timing effect is secured by a group of ram-operating valves with cams driven mechanically by a shaft from the bucket-lift mechanism. Thus the ram pushers will deliver the pouches to the belt at the proper intervals to be taken by the lift buckets when unloaded there. Co-ordination between the movement of the rams, the loading trays, the conveyors, and the lifts, is further secured by centering the motive power in one electric motor.

Outgoing Mail.—Mail in less than car-load lots will be handled directly into the cars by manual unloading from automobile trucks; these may reach the cars through the trucking subway and baggage lifts, or along the platforms from the Post Office lifts. Bulk and car-load mail, however, will be delivered into the cars automatically from the Post Office floors, as follows: The four central platforms, before referred to, adjoining the six car-storage tracks, are provided overhead with horizontal belt conveyors; these conveyors are placed in housings attached to the framework of the building, and west of it, in special

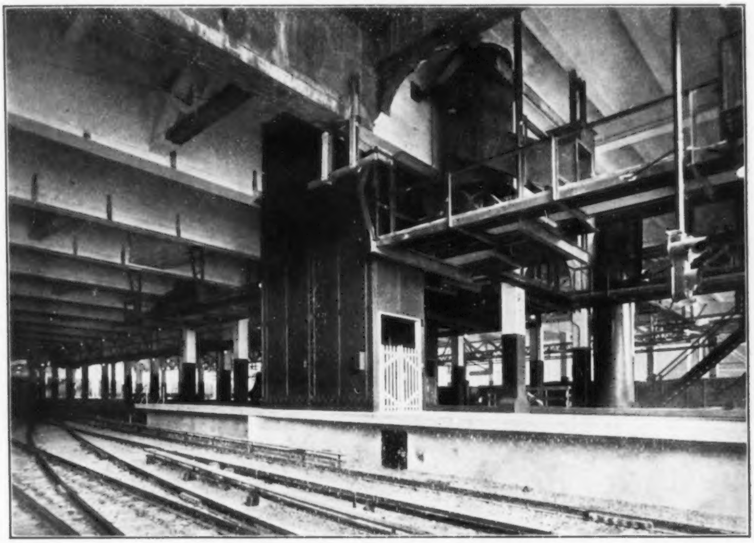


FIG. 1.—MAIL-HANDLING MACHINERY UNDER POST OFFICE.

FIG. 2.—SERVICE PLANT BUILDING.

structures carried by columns set in the platforms. The conveyor belts are driven by electric motors at a speed of about 100 ft. per min., and included in their paths are self-propelled carriages operating as "trippers" and running on tracks in the housings. These carriages may be run automatically to any desired position opposite the mail-car doors and operate to trip the mail pouch into a spout attached to the carriage and thence into the car door.

Mail from the working floor of the building is delivered to the belt conveyors through spiral chutes, single, double, triple or quadruple, as occasion requires, for simultaneous delivery from the various building floors and to either of the two conveyors east and west on each platform. Fig. 1, Plate LXXVII, is a view of the mail-handling machinery under the Post Office.

In addition to the conveyor system as described, there are four hydraulic lifts, of the plunger type, affording trucking communication between the building, the platforms, and the subways.

All the foregoing mail-handling facilities were provided by and will be operated by the Railroad Company. The special conveyor machinery was designed by Messrs. Marks and Woodwell, Consulting Engineers for the Architects of the Post Office, and was installed by the Lamson Belt Conveyor Company. The work was administered and co-ordinated in the usual manner through the office of the Chief Engineer of the Railroad Company.

Express Building.

The terminal operation does not at the present time include facilities for the handling of express freight matter; this class of business, for the time being, at least, will be cared for at the Jersey City Station of the Pennsylvania Railroad, and at Long Island City for the Long Island Railroad, as heretofore. To provide for the possible future handling of Express Company's freight at the new terminal, a location was set aside for an Express Building at the west side of Ninth Avenue, adjoining 31st Street. The area required by the building was excavated as part of the terminal yard, and, pending the erection of the building, will be used for a general storage yard. The area will admit of a building 130 by 180 ft., and the yard space under it will provide for one wide loading platform with a track at each side, accommodating a total of 12 express cars, with two additional

platforms in the yard between Eighth and Ninth Avenues, accommodating 24 cars. The total storage and loading capacity of the express yard tracks is 55 cars.

The express platforms will be reached from the building by hydraulic lifts, communicating with the main platform under it, and thence to the trucking subway system under the tracks and up to the other two platforms, and if desired, continuing through the subway to the main Station Building.

Service Power Plant.

Aside from the traction requirements, there are numerous and important uses of power in various forms in a large Station, namely:

(*a*) Heating and lighting the Station and other buildings;
(*b*) Steam, compressed air, and water supply for cars;
(*c*) Air supply for the signal system, for tunnel drainage pumping, and for sewage ejectors;
(*d*) Water supply for various purposes in the buildings, and for fire protection;
(*e*) Hydraulic power for elevators and lifts;
(*f*) Refrigeration for cold boxes in kitchens and restaurants, and for drinking water;
(*g*) Electric power for lighting buildings, tunnels, and yards; stationary motors for elevators, heating and ventilating fans in building and tunnels; motors for pumping in yard and tunnels; power for car battery charging, operation of telephones, clock system, and other minor uses;
(*h*) Traction power for moving trains.

Building.—The proper housing of all these important facilities required a large amount of space. The character and location of the Station precluded their installation in that building, and no central space for a special building was available in the yard without sacrifice of valuable track room. Fortunately, a convenient location was available on relatively cheap property of the Company on the south side of 31st Street, about midway between Eighth and Seventh Avenues, and directly accessible, under 31st Street, from the Station and yard. The building erected on the plot has a frontage of 160 ft., a depth of 95 ft. and a height of 86 ft. above the curb, with a depth of 49 ft. below. The building (Fig. 2, Plate LXXVII) was designed of a height and character to harmonize with the Station Building, with an endeavor to maintain the standards of the Station in all Company construc-

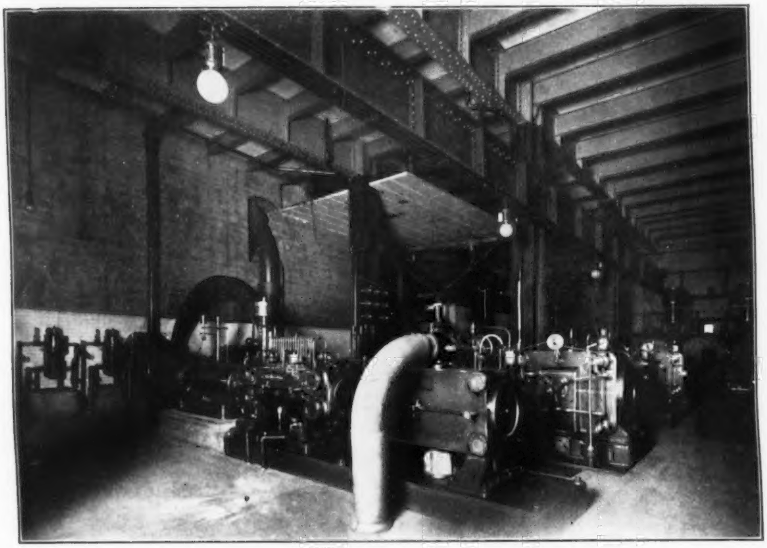


FIG. 1.—MAIN AIR COMPRESSORS IN SERVICE PLANT.

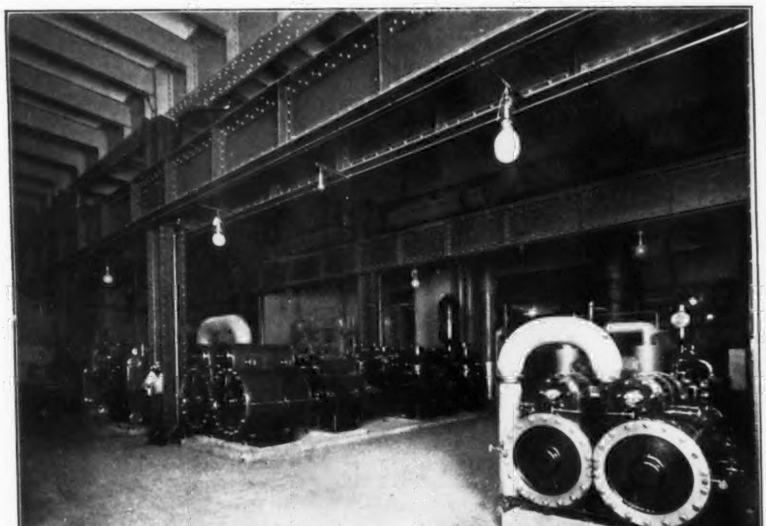


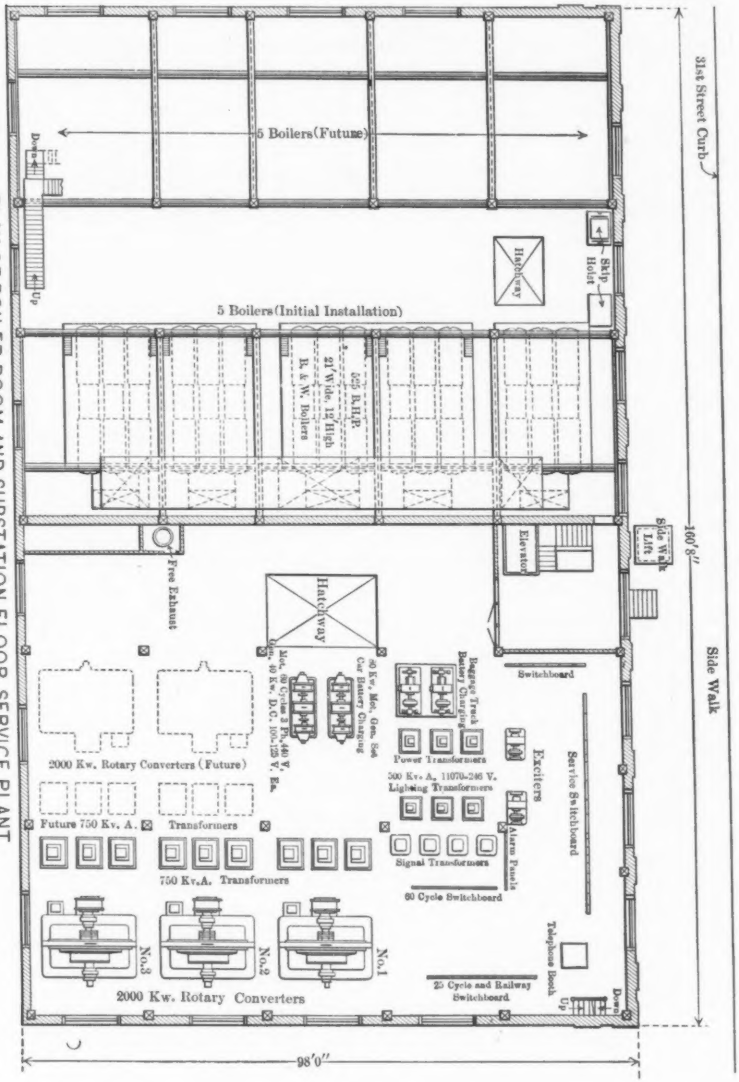
FIG. 2.—STEAM TURBO-GENERATORS IN SERVICE PLANT.

tions. The façade was designed by Messrs. McKim, Mead and White, and is of Stony Creek pink granite, similar in effect to that used for the Station exterior. The building construction is fire-proof throughout, having steel framing, masonry walls, and concrete floors and roof. The machinery on the various floors, and the coal storage and stacks required very heavy framing, 2 500 tons of steel having been used in the structure. The general plan, as shown by the drawings (see Fig. 7), divides the building by a fire-wall vertically into two main parts, the west half being devoted to the machinery and boiler plant, and the east to the traction sub-station, offices, and store-rooms.

The basement, which is at the level of the sub-grade below the tracks, contains the foundations for machinery, the engine piping, garbage destructor, cable-splicing chambers, and the main entrance to the pipe and trucking subways under the yard. The engine-room on the floor above (see Fig. 8) contains the lighting generators, the air compressors, the hydraulic power pumps, refrigerating engines and apparatus, and the heaters and circulating pumps for the Station. The third floor contains the fire and boiler pumps, water-storage tanks, and a pipe gallery to the Station Building. The next, or street floor, contains the boiler-room, above which are floors for coal storage, fan- and economizer-rooms. A brick stack terminates above the roof. Plate LXXVIII shows the main air compressors and the steam turbo-generator units.

The east half of the building (see Plate LXXIX) contains the traction sub-station, and has three floors below the street, for the cable inlets, bus-bar structures, and switching apparatus, and a main street-level floor for the rotary and switch-board room (see Fig. 1, Plate LXXX). Two store-room floors are above this, and the top floor is fitted up for offices to be used by a portion of the operating staff.

Boilers.—Space has been provided for ten 525-h.p. water-tube boilers arranged in two rows of five. Each row is provided with an independent draft system, smoke-flue, economizers, and stack. The initial installation consists of five of these boilers on the east side of the room; this equipment is sufficient to provide for the heating load of the buildings erected up to the present time, and of the yard, on the basis of an estimated load of 2 500 b.h.p. in the coldest day of the winter. When the proposed express and other buildings around the yard are erected, additional boilers will be installed as required.



PLAN OF BOILER-ROOM AND SUBSTATION FLOOR, SERVICE PLANT

FIG. 7.

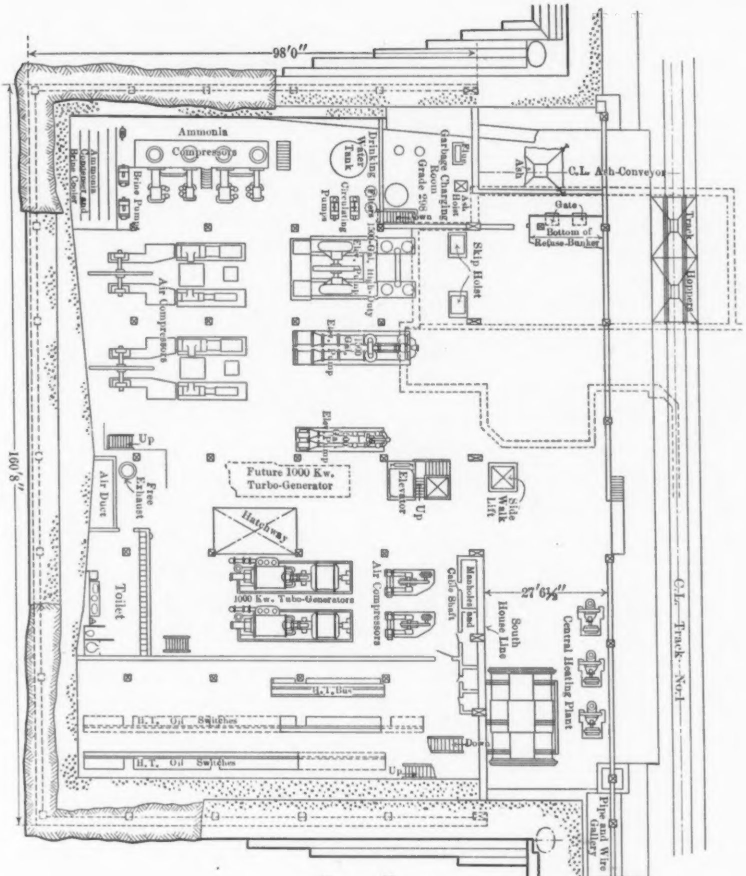


FIG. 8.

PLAN OF
SERVICE PLANT
AT
ENGINE ROOM
LEVEL

The boilers are arranged with shaking grates, for hand-firing with small sizes of anthracite coal. Draft is produced by a short stack supplemented with forced draft under the grates, furnished by fans of the Sirocco type, with 70-in. wheels, direct-connected to steam engines. The stack is of brick from its base at the roof of the building, and is carried on steel framing from the basement. It has an inside diameter of 11 ft., and is at present 50 ft. in height above the roof, or 127 ft. above the grates. If tall buildings at some future time adjoin the power-plant, the supports will admit of carrying the stack 150 ft. above the roof.

The ashes from the boilers are dumped into concrete-lined hoppers, arranged with the necessary gates to discharge into hopper cars which are moved on tracks to an ash-storage bunker of 80 tons capacity under the 31st Street sidewalk. From this bunker the ashes are carried by a belt-driven conveyor to railway cars standing on the southernmost track of the terminal. The conveyor is arranged so that it may be extended over the car when in use.

Coal is delivered by car to the track in front of the Service Plant, is dropped into a hopper under the track, and elevated by a belt conveyor discharging into feeding hoppers in the basement; from these it is hoisted in skips and discharged into the coal bunker by belts. This bunker is above the boilers, and has a storage capacity of 1 000 tons, or about one week's supply.

Two economizers, each of 1 050 tubes capacity, are provided for each row of five boilers, and located above the boiler-room. The boiler feed-pumps are of the compound duplex type, each pump having sufficient capacity to take care of the entire load under maximum conditions.

Water Supply.—Water for all purposes in the Service Plant, Station, and yard, is obtained from two private 12-in. mains in 31st Street, connected to independent city mains at Seventh and Ninth Avenues, respectively. The water is metered at the building and distributed through piping systems, as later indicated. As an emergency supply in case of a complete shut down of the city mains for a considerable period, two storage tanks, having a combined capacity of 75 000 gal., have been installed. The water system provides for:

(1) Boiler feed;
(2) Fire protection;

CROSS-SECTION
OF
SERVICE PLANT

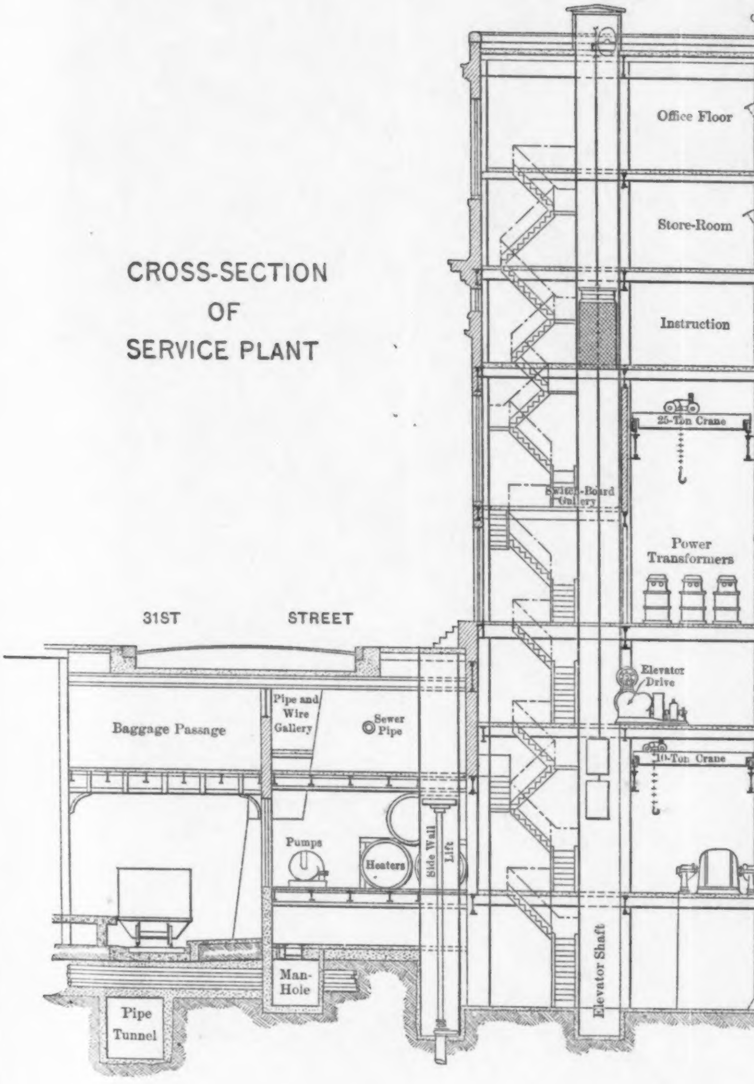
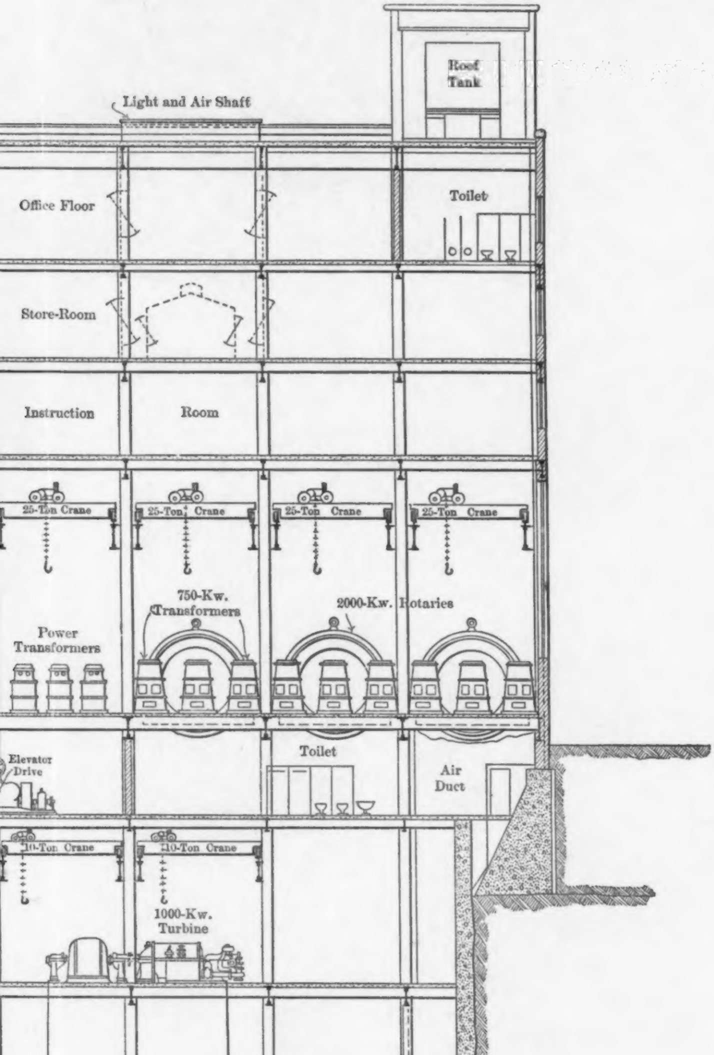


PLATE LXXIX.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.



(3) Fresh water for the general services in connection with kitchens and restaurant, and for lavatory purposes;

(4) Water for cooling air-compressor jackets; inter-coolers and after-coolers for the ammonia condensers, which is afterward used for flushing toilets;

(5) Water for general yard purposes, such as car cleaning.

Water for all purposes is drawn from the mains and storage tanks by the pumps, listed under the plant apparatus. There are two fire pumps, of the "Underwriters'" pattern, and they are kept constantly under steam, working slowly through regulators set to maintain a constant pressure of 90 lb. per sq. in., and drawn from to a limited extent for yard purposes. They are cross-connected to work singly or in battery.

For house-service purposes there are four duplex plunger, steam-driven pumps, having a capacity of 300 gal. per min. Two of these pumps (one being in reserve) are used for flushing-water for the toilet fixtures in the Station, and obtain their supply from the cooling water of the refrigerating plant and air compressors, which deliver into a 5 000-gal. storage tank on the roof of the Service Plant. Two pumps (one being in reserve) are used for pumping fresh water into a second 5 000-gal. storage tank on the roof, and furnish fresh water for various purposes in the Station, for which the cooling water used for flushing would not be suitable, such as for kitchen and restaurant purposes. These latter pumps deliver into two 1 500-gal. tanks on the roof of the Station Building over the kitchens.

For operating the hydraulic elevators and lifts in the Station and Post Office Buildings, three pumps have been provided; one 1 500-gal. high-duty, crank-and-fly-wheel pump, one 1 500-gal. compound, duplex, direct-acting, and one 500-gal. compound, duplex, direct-acting. These pumps are shown on Fig. 2, Plate LXXX. These sizes were selected after a study of the operating conditions during the busy-hour schedule, the maximum estimated quantity of water required at such time being 2 000 gal. per min. This requirement is met by operating one of the large pumps and the small one. During the winter, when exhaust steam is needed for heating, the direct-acting pump will generally be used, the high-duty pump being used when the demand for exhaust steam is light. These pumps operate at a steam pressure of 150 lb., against a back-pressure of 2 lb. from the exhaust, and furnish water at 300 lb. gauge pressure with a back-water pressure of 40 lb.

The heating system for the Station Building, elsewhere described, requires heaters for the water, and pumps for circulating the hot water. There are three heaters in the Service Plant. They are steel drums, $\frac{1}{2}$-in. thick, 5 ft. 7 in. in diameter and 23 ft. long. Each contains 557 2-in. wrought-iron tubes, giving 5 830 sq. ft. of heating surface. Steam passes through these tubes, and the water circulates around them, with suitable baffle-plates to secure the maximum efficiency of the heating surface. Each heater is piped to a water-circulating pump of the centrifugal type, direct-driven by 50-h.p. induction motors.

Air Compressors.—Compressed air is required for operating switches and signals, brake-testing in the yard, pumping in the tunnels, and for the sewage ejectors and the air cleaning machines. These various services were estimated to require a rated compressor capacity of 1 900 cu. ft. of free air per min. As exhaust steam is required for a large part of the year for the Station heating, it was found more economical to install steam- than motor-driven compressors. Therefore, two were provided, each capable of supplying the maximum total service demands. The machines are of the cross-compound, two-stage, Nordberg-Corliss-valve type, each having a capacity of 2 000 cu. ft. of free air per min., and compressing to 90 lb. per sq. in. The compressed air is passed through an atmospheric after-cooler, located on the roof of the building. In order, however, to provide for the higher pressure required for testing air brakes on cars, there is a separate compressor plant consisting of two two-stage, tandem-cylinder machines, each having a capacity of 100 cu. ft., and driven through link belts by 30-h.p. induction motors. These compressors deliver air at a pressure of 125 lb. per sq. in. to a separate piping system in the yard.

Refrigerating Plant.—Refrigeration is required for the cold boxes in the kitchen and dining-room department of the Station, and for cooling drinking water for fountains in various public rooms and corridors. The maximum requirements for all these purposes is equivalent to the melting of 56 tons of ice per day during the summer, or an average of 40 tons throughout the year.

Much consideration was given to the question of the most convenient and economical means of providing the required refrigeration for all terminal railroad purposes. The cooling of stationary boxes by ice would require the regular delivery of large quantities of ice to a great number of separate places, at considerable inconvenience, and

FIG. 1.—SWITCH-BOARD ROOM IN SUB-STATION, 31ST STREET SERVICE POWER PLANT.

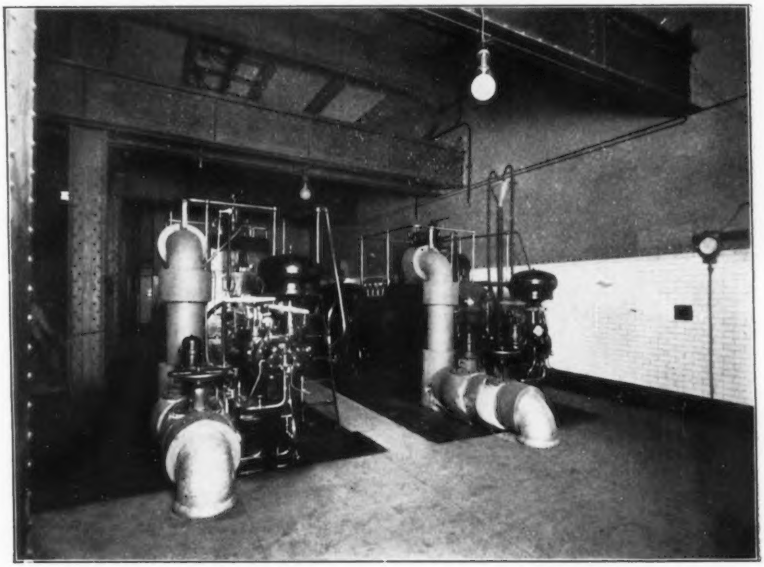


FIG. 2.—HYDRAULIC ELEVATOR PUMPS IN SERVICE PLANT.

it was calculated that a refrigerating plant would furnish the requirements at a lower cost, when installed in the Service Plant, together with the other facilities of that building. Ice, however, must be supplied in limited quantities for the cars and for drinking-water in the restaurant. For these latter purposes it was thought best to bring ice in cars through the tunnels, and deliver directly from the car in the yard, or from an ice-storage room provided in the Service Plant.

For the general refrigeration, however, a complete plant was installed, consisting of two units each of 40 tons capacity, one unit being generally available as spare. These circulate cooled brine through a piping system to the Station Building. The temperatures required in the cooling boxes throughout the building vary from a minimum of 10° Fahr. in the ice-cream box, to 28° Fahr. in the meat boxes, in order that the outgoing brine from the refrigerating system would leave the cooler at a temperature as low as 15 degrees. Two brine-circulating pumps have been installed, each being sufficient to circulate the required brine. In order to utilize the available floor space, vertical compressors were adopted, direct-driven by horizontal engines. They are single-acting, compression being accomplished during the upward stroke, and thus bringing only low evaporating ammonia pressure on the piston rod and piston packing, and minimizing the ammonia leakage. Each compressor is rated at 40 tons capacity on the basis of 185 lb. gauge, and an evaporating ammonia pressure of 15.67 lb. gauge. The engines have Corliss gear, operate at 76 rev. per min., and have a capacity of 70 i.h.p. The condensers are common to both the compressors, and are of about 50 tons capacity, with room to extend them later if found necessary. They are subdivided into sections, 12 pipes high, and are controlled by valves, to allow of convenient cleaning and repairs without interruption to the service. The interior arrangement consists of a horizontal, double-pipe system, made up of 2-in. outer pipes, carrying ammonia gas, and 1¼-in. inner pipes, carrying cooling water. The pipes are 20 ft. long, and their ends are joined by return bends and arranged for counter-current circulation of ammonia gas and water. The brine cooler consists of a nest of double piping similar in arrangement to the condenser. The outer pipes, which are 3 in. in diameter, contain the ammonia gas, and the 2-in. inner pipes, the brine circulation.

Drinking Water.—The apparatus comprising the drinking-water system is in the Service Plant, and consists of meters for measuring

the water from the city mains, a filtering plant having a capacity of 400 gal. per hour; a cooling and storage tank, having a capacity of 700 gal., operating in connection with the refrigerating plant described in the previous section, and two motor-driven, centrifugal pumps, delivering the cooled and filtered water to the special piping system for the fountains throughout the Station. The filtering is done alternately in two cast-iron, porcelain-lined drums, first through sand and then through charcoal. The current in these drums is reversed when required for cleaning. The water-cooling tank is of $\frac{1}{4}$-in. steel plate, enameled inside with glass enamel $\frac{1}{16}$ in. thick. The water is cooled in this tank by direct-expansion, ammonia-pipe coils, forming a part of the general refrigeration plant.

Garbage Destructor.—It was estimated that about 5 tons of wet kitchen garbage and the same quantity of dry refuse would accumulate in the terminal per day. Its prompt disposal was arranged for by the installation of an incinerating plant in the basement of the Service Building. This destructor is of the Morse-Boulger make, and consists of furnaces lined with fire-brick and containing four separate grates. In one of these the dry refuse (or coal) is used for fuel to supply heat for drying out and burning the wet refuse, and a second fuel grate is provided for an auxiliary fire to consume the gases from the offensive refuse. Two fire-brick grates, an upper and a lower, are used for wet garbage, which is dried and consumed by the heat from the fuel grates. The gases pass from the furnace to the main boilers and thence to the stack.

Garbage is brought from the kitchens in closed wagons to a cold room in the building, where it may remain until the furnace is ready for charging. It is then dumped on a tray on the floor above, sorted for valuable articles, such as silverware and linen, and then charged into the furnace through a grating at the top.

Lighting and Auxiliary Power Generators.—The approximate electric power requirements for all purposes (except for traction) are as follows:

Station lighting	600	kw.
Tunnel lighting	125	"
Motors for ventilation and station elevators	1 325	"
Motors for sump pumping	800	"
Motors for tunnel ventilation	350	"
Motors for Post Office mail-handling machinery	55	"
Power for signal system	175	"

All these various requirements might conceivably be met by the use of one source of electric power, and it would be possible to take the current from the main traction generators; but it can readily be seen that the services are so important that they cannot be considered as incidental, and therefore, the power system selected for them should not sacrifice perfect reliability and adaptability to the utmost degree of simplicity and economy in installation, or even of operation. The following considerations, therefore, were the guiding ones in designing this miscellaneous power system.

First.—It was determined to light the Station Building by small individual lights, rather than by powerful arc lamps; and, for reasons of economy, as elsewhere explained, it was decided to use Nernst lamps for the general lighting, a form of lamp requiring, for economy, alternating current at 240 volts.

Second.—The tunnels require to be lighted at all times by a system which is independent of the traction power, so that in case of failure of this latter, there will still be lights outside of the trains. The kind of current used for Station lighting will likewise be suitable for the same purpose in the tunnels.

Third.—Electric power is needed for operating the signal system, as elsewhere explained, and must, in part, be of the alternating-current type; the lighting generators, therefore, are suitable for the needed supply.

Fourth.—Alternating current may also be used for the various stationary motors, except that in case of the electric elevators, alternating-current motor control has not been as thoroughly perfected as in the case of direct-current control. Therefore, it was decided to operate all motors, except those for electric elevators, from the same alternating-current generators as were used for other miscellaneous power purposes. In order not to introduce another system of generation for electric current, it was decided to operate the elevators directly from the traction circuits at 650 volts.

Fifth.—For the minor purposes of car-battery charging, telephones, clocks, etc., small motor generators are used, converting the alternating into direct current of the required voltage.

The next consideration was that of properly safeguarding the generation of miscellaneous power, with due regard to economy in first cost and operation. As the Service Plant is essential for heating and

other yard purposes, it seemed, logically, the proper one to furnish auxiliary electric power, as the exhaust steam from the engines could be utilized in cold weather to provide heat for the Station Building. As laid out, therefore, the miscellaneous electric power system consists of two independent sources of power generation, with duplicate machines at each source; and machines of the alternating-current type are used, permitting distribution to the various services at any desired voltages, through suitable transformers. Fig. 1, Plate LXXXI, shows the remote-control traction feeder circuit-breakers and switch panels, in the Service Plant Sub-station; Fig. 2, Plate LXXXI, is a view in one of the pipe subways.

The following is a list of the variety of electric currents used for different miscellaneous power purposes, and produced at the Service Plant, except current for elevators, which is taken from the traction circuits; these are from one source, transformed and converted as required:

11 000-volt,	A.C.,	60-cycle,	High-tension service feeders.
2 200 "	"	"	Signal lines.
220 "	"	"	Signal lines in yard cabins.
650 "	D. C.	....	Third-rail feeders and office elevators.
420 "	A. C.,	60-cycle..	Motors.
240 "	"	" ..	Lights.
220 "	D. C.	....	Elevator control.
110 "	"	....	Car-battery charging and exciters.
35 "	"	....	Baggage-truck battery charging.

The machines installed to care for this general system, consist of two 1 000-kw. turbo-generators. These supply, through step-up transformers, three-phase alternating current, at 11 000 volts and 60 cycles, to the service power switch-board. They operate non-condensing, the exhaust steam being passed through the heaters for the Station Building heating. At seasons when the exhaust cannot be utilized for this purpose, the machines are shut down, the power supply being taken from generators in the Long Island City Power-House, where condensing turbines of a more economical rating are available.

Traction Sub-Station.—The traction sub-station is described in full under the general power heading, and need only be referred to

FIG. 1.—REMOTE-CONTROL, TRACTION FEEDER, CIRCUIT-BREAKERS AND SWITCH PANELS, IN SERVICE PLANT SUB-STATION.

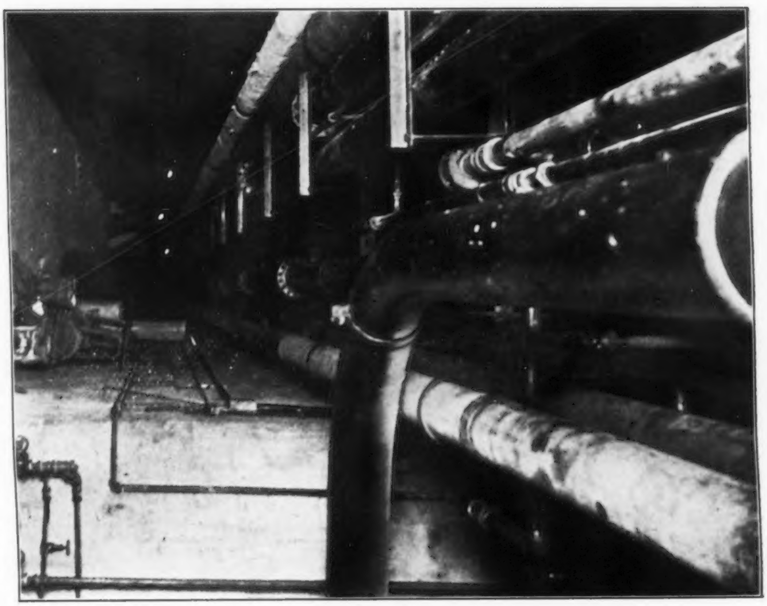


FIG. 2.—INTERIOR OF ONE OF THE PIPE SUBWAYS UNDER THE TRACKS.

here as consisting of the complete sub-station, occupying the east half of the Service Building.

Offices and Store-Rooms.—Above the main floor of the east half of the building are located the following:

(*a*) On a mezzanine floor overlooking the rotary-room, a space of 980 sq. ft., for locker-room and toilets, and offices for the service plant foreman and assistants;

(*b*) On the third floor an instruction-room and office, 6 200 sq. ft. in area, containing apparatus to illustrate the construction and methods of operation of the special devices of the signal system, electrical equipment of the division, etc. These include complete working parts of the devices in question, with sectional models showing their internal construction. The systems illustrated are:

Block and interlocking signals,
Automatic train stop,
Car and locomotive air brakes,
Electric control apparatus of locomotives and cars,
Tunnel alarm and telephone boxes,
Third-rail construction and switching apparatus,
Apparatus for testing sight and hearing.

(*c*) The fourth floor, an area of 6 200 sq. ft., has been fitted for a general store-room for supplies of all kinds;

(*d*) The fifth floor is subdivided into offices for certain of the division staff; the space for offices, excluding corridors, being 4 300 sq. ft.

The following is a concise statement of the Service Plant machinery equipment, excluding the traction sub-station equipment listed elsewhere:

Water-tube boilers, 525-h.p. each; ultimate capacity, ten boilers; pressure, 200 lb.	5
Single-acting, high-speed, steam engines, 50 h.p. each, driving Sirocco fans, for forced draft	2
Green fuel economizers	2
Water storage tanks; capacity 60 000 gal	4
Coal conveyor, two pairs, capacity 120 tons per hour	4
Coal-skip hoist engine	1
Ash conveyor, 50 tons per hour, motor-driven, belted	1
Garbage destructor	1

Boiler feed-water heater; capacity, 6 500 gal. per hour from 70° to 200° Fahr., 300 gal. per min.	1
Boiler feed-pumps, duplex-tandem, compound, steam-driven.	2
Hot-well pumps, centrifugal, motor-driven.	2
1 000-kw. 240-volt, 3-phase, 60-cycle Westinghouse-Parsons, steam-driven, direct-connected turbo-generators. (Space for one additional)	2
2 000-cu. ft. per min., Nordberg, cross-compound, Corliss, steam-driven, air compressors, 100 lb. pressure, for signals and sewage ejectors	2
100-cu. ft., motor-driven, air compressors, 120 lb. pressure, brake testing	2
Elevator pump, 1 500 gal. per min., 300 lb. pressure, steam-driven, for hydraulic baggage and passenger elevators.	1
Elevator pump, 500 gal. per min., 300 lb. pressure, steam-driven, for hydraulic baggage and passenger elevators.	1
Compound, steam-driven pump, 1 500 gal. per min., for baggage and passenger elevators.	1
Steam-driven, Westinghouse, air-brake pumps, for elevator system and for air cushion, hydraulic elevater tanks.	2
Duplex, steam-driven, Underwriters' fire pumps, capacity, 1 500 gal. per min. each.	2
Motor-driven, automatic, centrifugal pumps, for circulating cold water and water for flushing purposes.	3
Motor-driven, centrifugal pumps, for hot-water circulation to Station indirect heating system.	3
Heater for Station indirect heating system.	3
Heater, 635 gal. per hour, for hot water in Service Plant.	1
Refrigerating plant, 40-ton ammonia compressors, engine-driven.	2
Brine pumps, motor-driven, centrifugal.	3
Motor-driven, 1½-in. centrifugal pumps for circulating drinking water	2
750-kw., single-phase, 60-cycle, 11 000-420-volt, air-blast transformers	3
500-kw., single-phase, 60-cycle, 11 000-246-volt, air-blast transformers	3
150-kw., 60-cycle, 11 000-2 200-volt, O.I.S.C. transformers for signals	2
100-kw., 60-cycle, 11 000-220-volt, O.I.S.C. transformers for signals	2
80-kw., 110-220-volt, motor-generator sets, for car-battery charging	2
40-kw., 110-volt, motor-driven exciters.	2
25-kw., 35-volt, motor-generator sets, for baggage-truck charging.	2
55-cell storage battery, for emergency excitation.	1

Tunnel Facilities.

The extensive system of more than 15 miles of single-track tunnels has been fully described, as to physical construction, by others; the railroad and the operating facilities, however, will be referred to briefly.

Size of Tunnels.—The size of the tunnels was a consideration of great importance in the subaqueous tubes, where a determination of the minimum practical operating diameter was essential in order to avoid a large expenditure of money unnecessarily. In a tunnel of circular cross-section, the height from the invert to the crown of the arch is the controlling dimension, to accommodate the track structure and the cars. This height, therefore, was considered carefully by the Board of Engineers and the Committee of Operating Officers, having due regard to probable deviation from a true central axis in the process of tunnel driving. A minimum was fixed for the depth of the ballasted form of track, and a minimum clearance over the top of the cars in which to provide for wrecking operations, and also for the possible use of an overhead electric conductor for traction, should this later be considered practicable and desirable at the time when the traction system was determined on, or even at a later date, should developments in the art require it. The adopted clearances are given in the following table of tunnel data:

Total length of single-track tunnels........	15.54 miles
Total length of river tubes................	12.02 "
Inside diameter of tube tunnels...........	19.00 ft.
Distance from top of rail to crown of tube tunnel arch	16.00 "
Minimum top clearance over car roof to crown	1 ft. 9 in.
Distance from tunnel invert to top of rail...	2 ft.
Height of top of tunnel benches above rail..	5 ft. 6 in.
Width between benches....................	11 ft. 8 in.
Width of bench (River sections)...........	3 ft. 8 in.

Drawings giving full particulars of the tunnel dimensions are found in other papers. The average clearance above the car roof to the crown of the tunnel arch is 21 in., and it was intended, in the event of the use of an overhead conductor, to give a minimum clearance of 6 in. from the crown to the conductor and 21 in. from the conductor to the car roof. In order to obtain maximum space for the insulators carrying the conductor, it was decided to construct pockets in the concrete

of the tunnel roof, these being 15 by 24 in., 5 in. deep, and 10 ft. from center to center.

Interior Arrangement.—The circular cross-section of the tunnel provided an excess of clearance at the horizontal diameter, which President Cassatt suggested might be utilized to contain high side-benches of concrete, to form vertical walls for the protection of trains in case

CROSS-SECTION OF RIVER TUNNEL

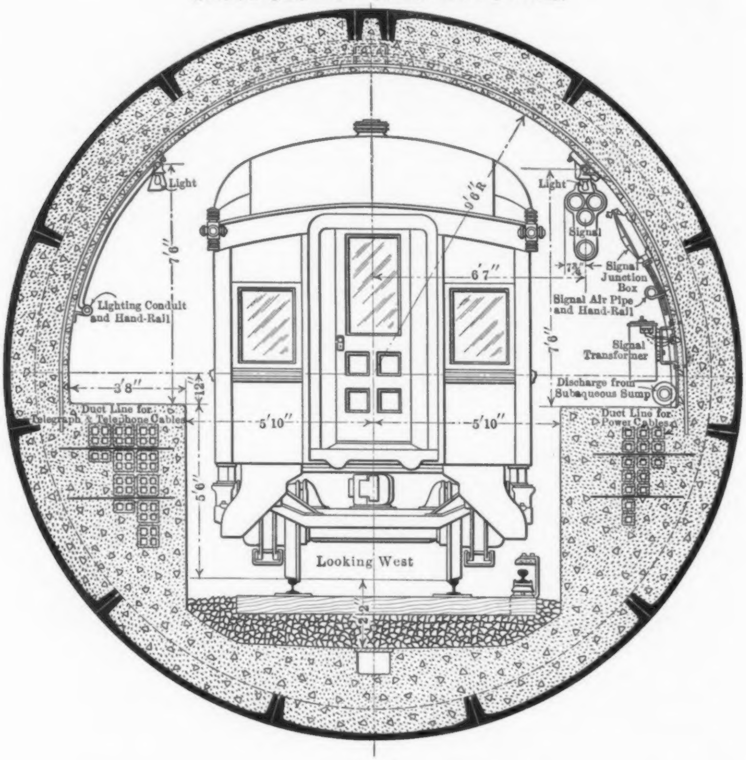


FIG. 9.

of derailment, and also for walkways for passengers in case of accident, and for employees. (See Fig. 9.) They also provide convenient and safe means for housing the electric cables. These benches were made with the top 5 ft. 6 in. above the rail, or about 18 in. above the car floor, and give normally 10 in. clearance from the sides of the car. Fig. 1, Plate LXXXII, is a view of the interior of the tunnel, showing the signal equipment and other facilities.

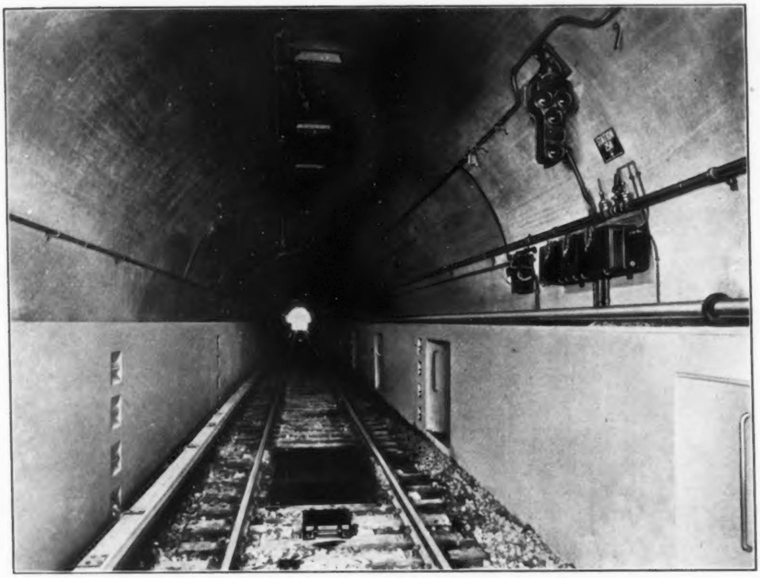


FIG. 1.—INTERIOR OF TUNNEL, SHOWING SIGNAL EQUIPMENT AND OTHER FACILITIES.

FIG. 2.—VERTICAL-SHAFT MOTORS FOR CENTRIFUGAL PUMPS IN NINTH AVENUE SUMP.

The walkways in the river tubes are 3 ft. 8 in. wide, but in certain portions of the land section their width is only 2 ft. 4 in., as required by the width of the streets traversed. In installing signals, piping, and other apparatus in the tunnels, only one bench (that on the right hand side) has been obstructed, and this only partly; the other bench has been left entirely clear. Both benches have been provided with hand-rails, and the tunnel lights are placed above them in a convenient location for inspection and renewal.

It will be noted, from other papers treating of the tunnel construction, that refuge niches and ladders for trackmen are provided in the benches at intervals of 25 ft. It will also be noted that the tunnels are provided with cross-passages between pairs in the land sections. These passages are closed by doors, so that the tubes throughout are distinct, and ventilation is positive.

Drainage.—The drainage is described under the general heading, "Drainage System."

Lighting.—The tunnels are lighted continuously by a source of power which is entirely independent of the traction system. Each tunnel, moreover, has two circuits which are independent of each other and are fed from separate transformers and switching panels. The primary source of the lighting current is the 60-cycle generators used for the general auxiliary power system, and located in the Long Island power-house and also in the 31st Street Service Plant.

The lighting circuits are run in pipe conduits on the tunnel walls above the benches, and on one side of the tunnel serve as a hand-rail. The lamps are of the Tungsten type, of 25-watt rating, 20-c.p., and operate at 33 volts, being connected 8 in series on a 252-volt circuit. They are 50 ft. apart, on each side of the tunnel, and staggered so as to give a lamp for each 25 ft. of the tunnel length. The lamps are 7 ft. above the tunnel side-benches, and have enameled steel reflectors to throw the light in the direction of the movement of trains. Arrangements are made at various points in the lighting circuits for the attachment of portable extension connections for lamps to be used in repair work on various apparatus in the tunnels. The control of the current is from switch-boards in the auxiliary power sub-stations in the shafts.

Ventilation.—Satisfactory ventilation for the tunnels was considered to be of great importance, as it was desired that, not only should

the tunnels be safe under all emergency conditions, but that there should be at no time noticeable discomfort to passengers. Two general conditions were to be provided for: first, purity of the air in normal operation; and, second, requisite ventilation for an indefinite period in case of stoppage of trains in the tunnel from accident or other cause. It was thought, and afterward verified by trial, that the piston action of the trains when in motion would be an effective means of changing the air, as each tube contains only one track, and is isolated from the adjoining tube and open at each end to the free air. Where piston action has proved insufficient, as in the case of the deep tubes in London, it would seem to be because of the lack of sufficient free opening to the atmosphere, especially at the ends, and because of the by-passing of the air from one tube to another at stations. A special ventilating system, therefore, is needed only to provide air to a stalled train in an emergency, or to dissipate smoke and fumes from an electric arc, the possibility of which conditions was thought of sufficient importance to warrant the installation of a very complete forced-draft ventilating plant. To obtain the benefit of his experience in tunnel ventilation, the Company engaged Charles S. Churchill, M. Am. Soc. C. E., as an expert to consult with the writer in devising a proper system for the 15½ miles of tunnels comprised in the terminal railway. It was determined that the air in the cars should not be allowed to contain more than 8 parts of carbon dioxide per 10 000, requiring 30 cu. ft. of fresh air per min. to each passenger. To insure this quantity of fresh air in the cars, it was thought advisable to furnish more, namely, 50 cu. ft. per passenger per min. in the tunnels, and the fan equipment was designed to meet this requirement, having due regard to emergency conditions and the occasional irregular spacing of trains. The quantity of air required per section of tunnel on this basis is about 60 000 cu. ft. per min., which will completely change the contents of the tubes three times per hour.

Plans for producing the requisite ventilation by exhaust, by pressure, or by a combination of both were considered. The system found best adapted to the local conditions was patterned after the one devised by Mr. Churchill, and used on the Norfolk and Western Railway and elsewhere. It is a forced-draft system in which a constant and uniform current of air is induced in the tunnel by forcing, in the direction of the traffic, the required volume of air into the portal. A

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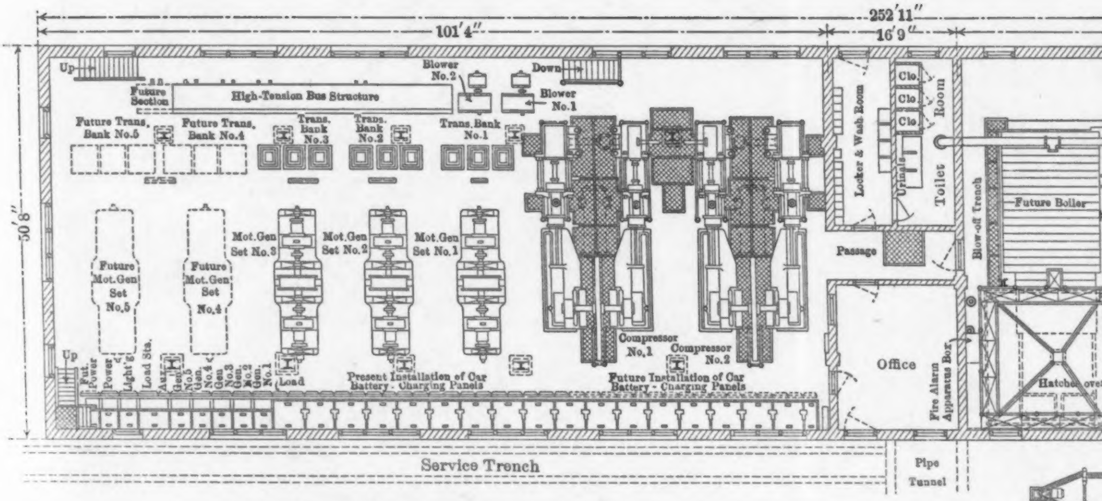


PLAN OF BOILER-HOUSE AND AUXILIARY SUBSTATION, SUNNYSIDE YARD.

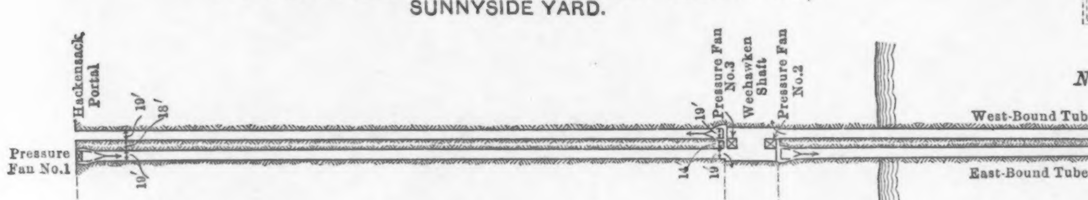


DIAGRAM OF TUNNEL VENTILATION S

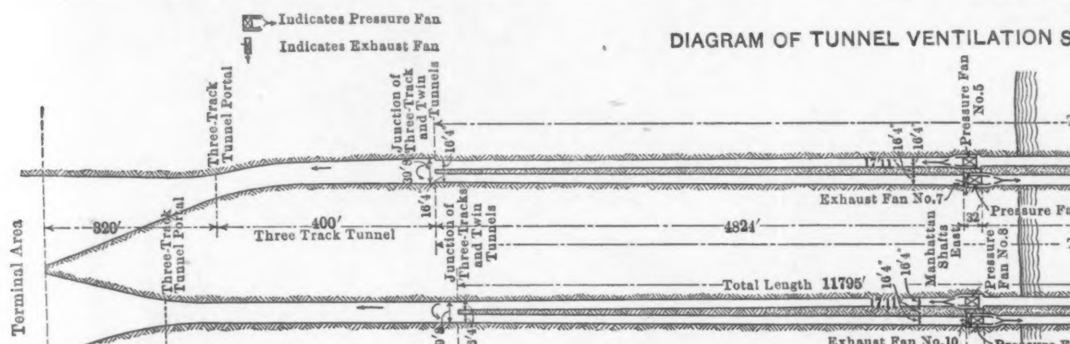
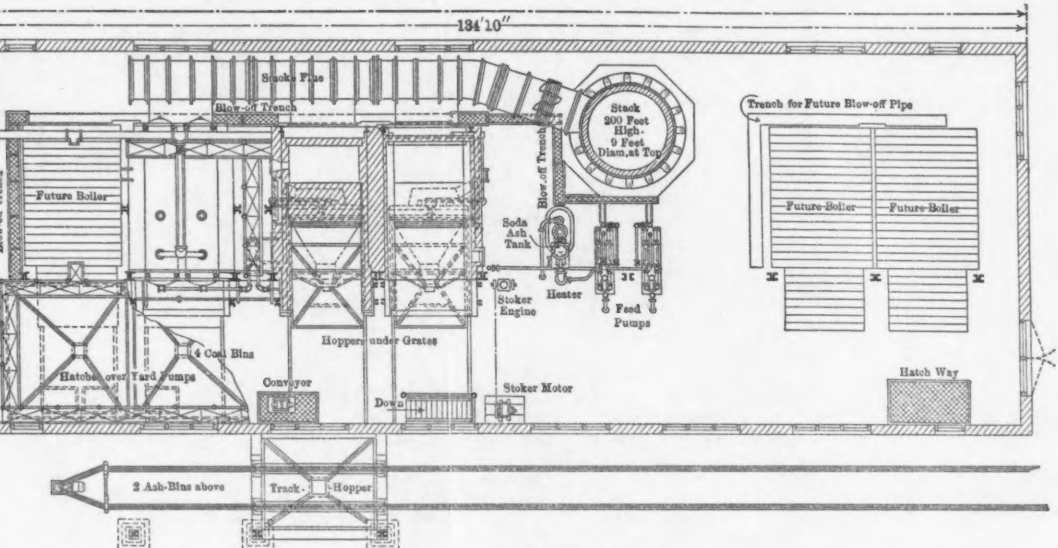
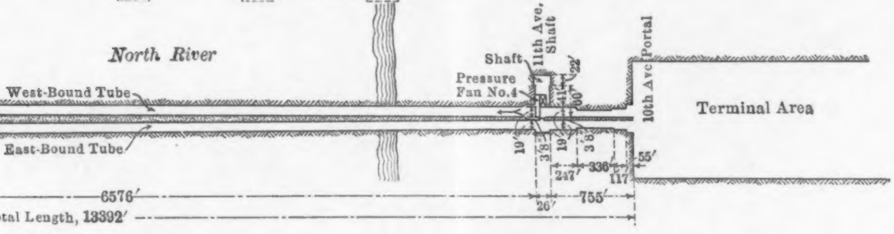
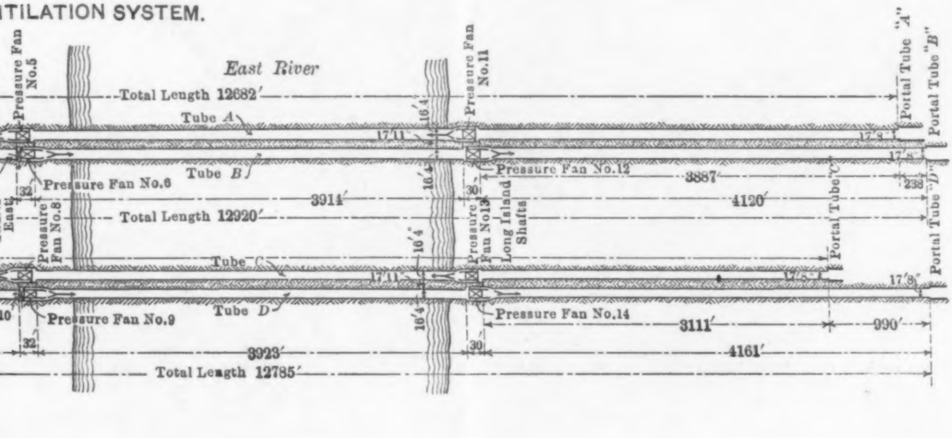


PLATE LXXXIII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.





TILATION SYSTEM.



divided nozzle, in the form of a tapering flue, is placed on each bench-wall for this purpose. This method requires no obstructing flues in the tunnels themselves, the nozzles being at the portals only, a consideration of great importance in keeping the side-benches free for walkways. The arrangement of tunnels and shaft openings required in all fourteen sets of ventilating apparatus at different points, as shown by the lower part of Plate LXXXIII. A list of the locations with fan capacities obtained by test under maximum conditions is given in Table 4.

TABLE 4.—TUNNEL VENTILATION.

Location.	Fan No.	Blower or exhauster.	Direction of air—east or west.	Diameter of wheel, in inches.	Approximate length of tunnel ventilated, in feet.	Fan capacity at normal speed, in cubic feet per minute.	Velocity at fan outlet, in feet per minute.	Brake-horse-power at motor.
North River—Hackensack Portal, Building over portal......	1	B	E	60	5 900	87 000	3 920	48.6
North River, Weehawken Shaft. In room between tracks below bench level...............	2	B	E	60	6 600	125 800	5 670	97.0
	3	B	W	54	6 000	100 800	5 580	56.8
North River—Eleventh Avenue Shaft. In room at track level on north side of west-bound tube...........................	4	B	W	60	6 600	107 000	4 830	68.5
East River—First Avenue Shafts. In building located at ground level over tracks 3 and 4..........................	5	B	W	72	4 800	100 000	3 130	62.0
	6	B	E	54	3 900	60 000	3 330	21.0
	7	E	E	66	4 800	59 200	4 400	27.9
East River—First Avenue Shafts. In building located at ground level over tracks 1 and 2..........................	8	B	W	72	4 800	100 000	3 130	62.0
	9	B	E	54	3 900	60 000	3 330	21.0
	10	E	E	66	4 800	59 200	4 400	27.9
East River—Long Island City Shafts. In building located at ground level over tracks 3 and 4..........................	11	B	W	54	3 900	65 000	3 600	24.3
	12	B	E	54	4 100	65 000	3 600	24.3
East River—Long Island City Shafts. In building located at ground level over tracks 1 and 2..........................	13	B	W	54	3 900	65 000	3 600	24.3
	14	B	E	54	4 200	65 000	3 600	24.3

It will be noted from Table 4 that, in two cases, exhausting instead of pressure blowers are used; these are for the purpose of causing a return current of air at the west ends of the cross-town tunnels; where they merge into three-track tunnels approaching the passenger station, the object being to prevent blowing air from the tunnels under and into the Station Building.

The blowers are of the multi-vane, "Sirocco" type, belt-driven from induction-type electric motors; and the speed of the fan can be ad-

justed, by cone pulleys, from normal, as given in Table 4, to 70% or 40% of normal, as required.

The air ducts from the fans (see Fig. 10) vary in form and arrangement to suit local conditions, but are generally rectangular in section. They were designed carefully, with bends of large radius to minimize friction, and proportioned so as to eliminate all sudden changes in velocity between the fan and the nozzle.

From tests made in the tunnels, with and without the fans running, it is apparent that, under normal conditions, the piston action of the trains can be relied on to give satisfactory ventilation. Records show that in the East River Tunnels the air is changed every 40 min. by the passage of trains during non-rush hours, and every 15 min. during rush hours.

The average velocity of the air in the East River Tunnels due to the action of the fans alone is about 8 miles per hour. This is increased by the passage of trains to more than 30 miles per hour, the latter figure, of course, depending on the number of cars in the train, and the speed.

It is evident, therefore, that in regular operation, the fans need not be run, and provision has been made to start and stop them, as required, from two central points, the power-house for the East River Tunnels, and the Service Plant for the North River Tunnels.

Tunnel-Alarm System.—The tunnels are equipped with a special safety device which has two functions, one to cut off the current in a given section of the third-rail, and the other to send a fire-alarm call. The system consists of a series of alarm boxes, set about 800 ft. apart. Each box is numbered, and contains two levers, colored blue and red, respectively. The blue lever is marked "POWER," and when pulled trips the circuit-breakers controlling the third-rail section adjacent to the box, thus cutting off the power and at the same time sending a call of two rounds of the alarm box number to the connected indicators. The red lever is marked "FIRE," and when pulled performs the same function as the power lever, but sends in two additional rounds of the box number.

The box mechanism is operated by clockwork, set in motion by the winding of a spring when the lever is pulled; the clockwork actuates electric contacts in the circuits, controls an auxiliary tripper to the section circuit-breaker, and spells the box number on the station

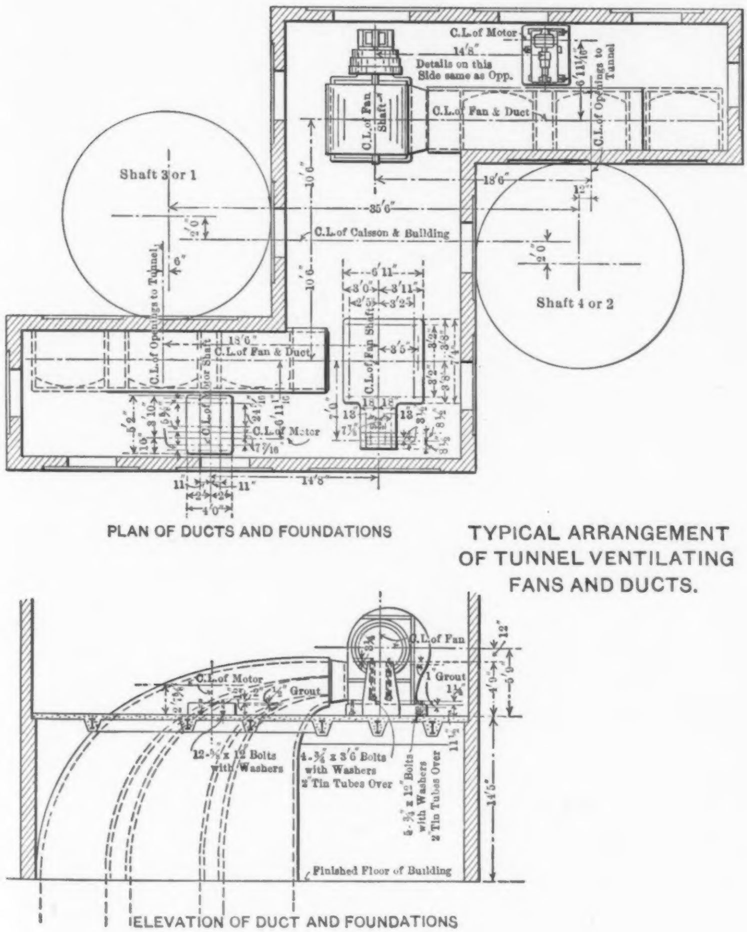

C.L. of Motor
Details on this Side same as Opp.
C.L. of Fan Shaft
C.L. of Fan & Duct
C.L. of Openings to Tunnel
Shaft 3 or 1
Shaft 4 or 2
C.L. of Caisson & Building
C.L. of Openings to Tunnel
C.L. of Fan & Duct
C.L. of Motor Shaft
C.L. of Fan Shaft
C.L. of Motor
PLAN OF DUCTS AND FOUNDATIONS
TYPICAL ARRANGEMENT OF TUNNEL VENTILATING FANS AND DUCTS.
C.L. of Fan
C.L. of Motor
Grout
1" Grout
12-⅝" x 12" Bolts with Washers
4-¾" x 3'6" Bolts with Washers 2" Tin Tubes Over
Finished Floor of Building
ELEVATION OF DUCT AND FOUNDATIONS

FIG. 10.

indicators. Each box is provided with an interference magnet which prevents sending in an alarm from another box if one box on the circuit is in operation. Current for actuating the alarm circuits is obtained from storage batteries in the various sub-stations.

TABLE 5.—TUNNEL ALARM SYSTEM.

General-alarm circuits.	Location.	Total No. of boxes in each tunnel.	Local alarm circuits. No.	No. of boxes in each local circuit.	Location of switch-board for local circuits.
No. 1, North River.....	South Tunnel..	19	1	9	Sub-station No. 3.
			2	10	" " 2.
	North Tunnel..	19	3	9	Sub-station No. 3.
			4	10	" " 2.
No. 2, East River.....	Tunnel No. 1...	20	5	4	Sub-station No. 2.
			6	9	Long Island City Power-Station.
			7	7	" " "
	Tunnel No. 2...	18	8	4	Sub-station No. 2.
			9	9	Long Island City Power-House.
			10	5	" " "
No. 3, East River.....	Tunnel No. 3...	20	11	4	Sub-station No. 2.
			12	9	Long Island City Power-Station.
			13	7	" " "
	Tunnel No. 4...	20	14	4	Sub-station No. 2.
			15	9	Long Island City Power-Station.
			16	7	" " "

The indicators, recording the character of the alarm and the location of the box sending it, are in the offices of the Train Director, the Train Despatcher, and the Power Director, and in the power substation controlling the traction current for the section in question; the indication is also repeated to the interlocking-switch cabin controlling train movements to the section. In case of a partial short circuit on a car or at the third-rail, which may maintain an arc, or in case it is desired to work around defective apparatus under a standing train, the current may be cut off from the third-rail of the section by pulling the "POWER" lever. In case of a serious fire or other emergency, the "FIRE" lever may be pulled, and then the Railroad Fire Department at the Station and the emergency crews will respond. In either case the person pulling the lever is instructed to get into direct communication with the Train Director by telephone from a near-by telephone box, and to explain the trouble so that an order governing subsequent procedure in responding to the alarm or resetting the circuit-breakers may be given.

There are sixteen local or "POWER" alarm circuits, corresponding to the section-controlling breakers, and there are three general or "FIRE"

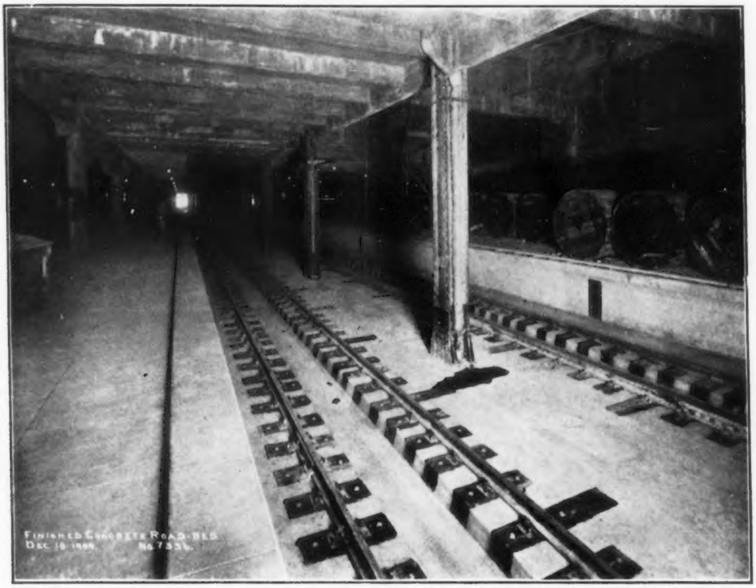


FIG. 1.—FINISHED CONCRETE TRACK.

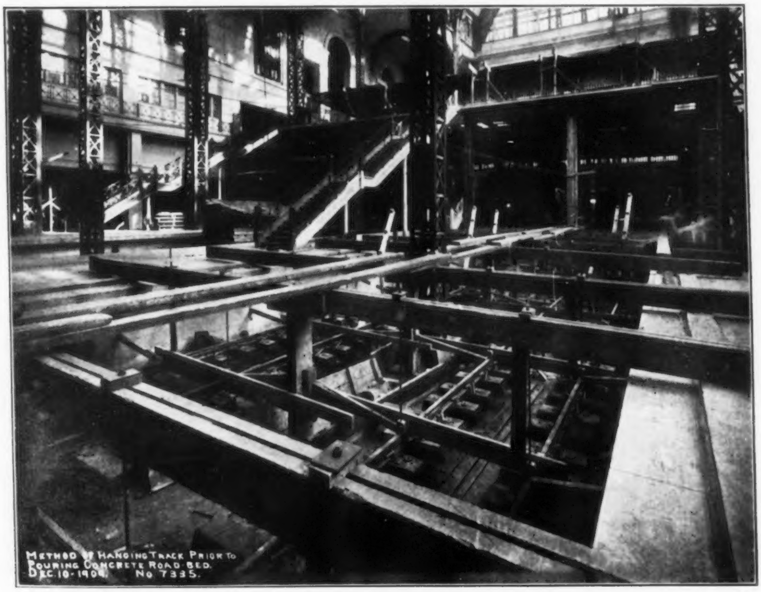


FIG. 2.—METHOD OF SUPPORTING TRACK STRUCTURE TO RECEIVE CONCRETE BASE.

alarm circuits, connected to switch-boards in the power-house at Long Island City, the Service Plant in 31st Street, and the traction substation at the Hackensack Portals of the tunnels.

Track.

The portion of the Terminal railway track construction assigned to this Department consisted of that in the tunnels, and in the Station and Sunnyside yards. The standards adopted are in general those of the Pennsylvania Railroad, with certain modifications in detail dictated by the Maintenance of Way Department for the better adaptation of the track to the peculiar local conditions.

Tunnel Track.—It was desired to adapt the track to high-speed running, with a minimum of vibration of the track or tunnel structure, and to reduce noise as far as practicable; also to permit of ease of renewal, without disturbance of the tunnel concrete lining. For these reasons, it was decided to adopt ballasted track, rather than any special form built into the tunnel structure, although, as mentioned later, a short section of track having a concreted base was put in the land section of two of the tubes, for experimental purposes.

The rail is 100 lb. per yd., and is of the new "Pennsylvania" section, and of open-hearth steel, to the Road's specifications. The joint angle-bars are of the six-hole type, with an extended flange below the rail base, and have 1-in. bolts. The base has also a special cross-section providing space for copper bonds between rail and splice.

The ties are of black gum and yellow pine, creosoted, the minimum dimensions being 8 in. face, 7 in. thick, and 18 ties to the rail length of 33 ft. All track is tie-plated with special rolled-steel plates 7 by 13 in. and $\frac{5}{8}$ in. thick. These plates have shoulders inside and out, and are secured to the ties by four $\frac{3}{4}$-in. lag-screws. Under each plate is placed a pad of compressed hair-felt, $\frac{1}{2}$ in. thick, the tie being dapped out to receive the pad. The rail is secured to the tie through the plate by two 1-in. lag-screws, each bearing on the rail flange and shoulder of the plate.

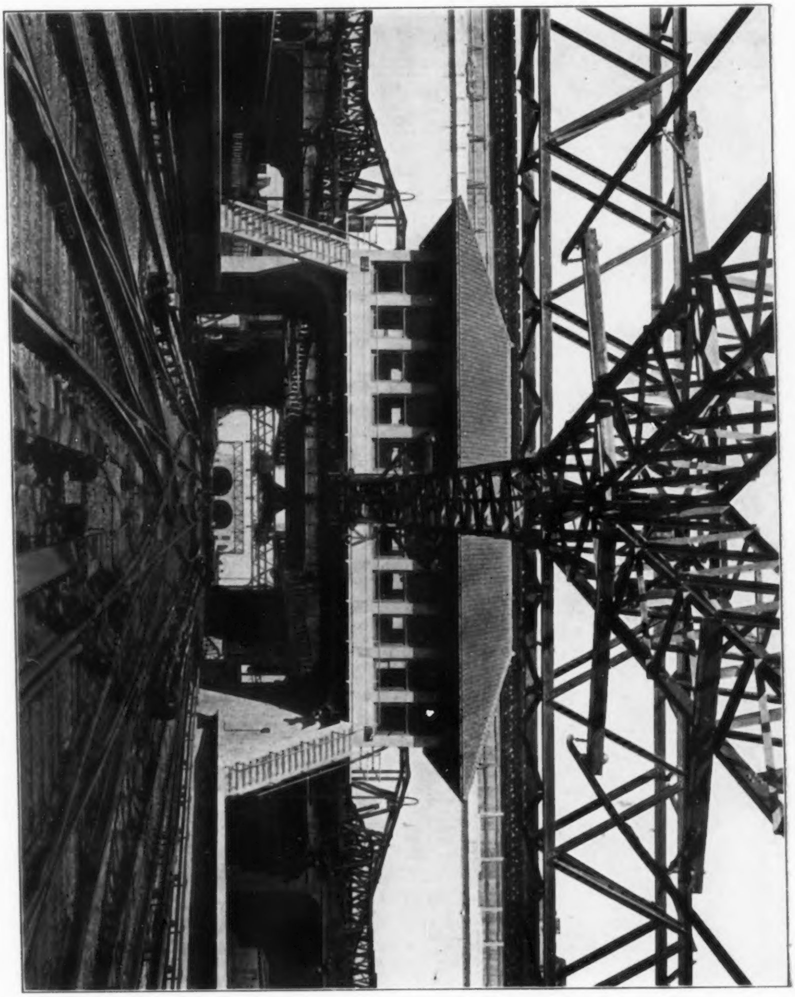
The ballast is of trap rock, screened through $1\frac{1}{2}$-in. mesh, and laid for a depth of 12 in. under the ties. This ballast was crushed on the Company's property, using the rock taken from the Bergen Hill Tunnels.

Station Yard Track.—Two types of track construction were adopted for the yard (see Fig. 2); one, the standard ballasted, and the other a special form of concrete bedding for the short tie-blocks which carry the rails. Ballasted track is used through the switches and in the storage yard, and the concreted-base track adjacent to the platforms. The ballasted track is of the same standards as used in the tunnels, except that the ballast is of $\frac{3}{4}$-in. rock, and the ties are not creosoted.

Concrete-Base Track.—For the track under the Station Building and adjacent to the passenger platforms, it was desired to provide a form of construction which would present a smooth surface, and could readily be kept clean. This was especially desirable where cars stand and drip oil and water on the track structure, and at places where rubbish may be thrown on the tracks by passengers. To devise the proper form of track for this purpose, a special committee was formed of Operating Officials of the Pennsylvania Railroad. This committee recommended a special form of track laid on wooden blocks embedded in a concrete base. This form of construction is shown in the lower part of Plate LXVII, and in Fig. 1, Plate LXXXIV. A single track length of 14 600 ft. of this type was laid adjacent to the platforms, except at switches and cross-overs, where standard ballasted track was used. In general, the concrete surface was laid on the rock of the sub-grade, but in places where the sub-grade consisted of loose rock back-filling, crossed by drains and subways, it was necessary to secure uniformity for the concrete base by specially ramming the filling and using rod reinforcement or bridging in the concrete. The concrete used is a 1:2:4 mixture of Portland cement, sand, and washed Cow Bay gravel.

The track was first laid complete with its fastenings to the tie-blocks, and then raised and leveled with great care to the proper grade and alignment by hanging the structure from a timber bridging carried on the station platforms as shown by Fig. 2, Plate LXXXIV; the concrete mixture was then poured, tamped, and allowed to set; then the bridging was removed. The cost of this track complete, under the special conditions obtaining in this place, was \$6.27 per lin. ft. for the base, and \$2.67 per lin. ft. for the tie-blocks and fastenings, making a total cost of \$8.94 per lin. ft.

In the Station area, of course, this track is subjected to slow-speed running only. In order to test its application to high-speed running,



SIGNAL CABIN "A," LOOKING TOWARD TUNNEL PORTAL AT TENTH AVENUE.

two short sections, each of 720 ft., were laid in two of the East River Tunnels (No. 1 and No. 2), immediately east of the Long Island shafts. In this case the concrete base for the track was laid directly on the concrete invert of the tunnel lining, and the general methods of construction were the same as described for similar track in the Station yard.

Sunnyside Yard Track.—The main running tracks are laid with 100-lb. rails, on oak ties; the yard tracks are of 85-lb. rails, on untreated yellow pine ties. All rail is placed on tie-plates, without the hair-felt pad beneath, and the rail and plates are secured to the ties by standard ⅝-in. spikes.

Third-Rail Ties.—Each fifth tie in the track is used at one end as a seat for the third-rail insulator and for the bracket to support its covering; therefore, they were longer than the standard ties, or 9 ft. 3 in.

Frogs and Switches.—All frogs and switches are according to the Pennsylvania Railroad standards; generally No. 8 frogs are used and 18-ft. point switches, housed in the stock rail; in a few cases clearance has required the use of a No. 7 turn-out. In the Station yard all frogs and crossings have hardened manganese steel points. The general quantities relating to track are as follows:

Total length of track laid, including Meadows Division and Manhattan Transfer yard	94.71 miles
Ballast used	268 870 cu. yd.
Number of switches	357
Number of slips	46
Number of crossings	14

Buildings for Railroad Facilities.

Sixty-four separate buildings were required for the Terminal Railroad. They are listed in Table 6 as to location and purpose. The details of design and construction of these various buildings differ greatly, and cannot be described with any fullness in this paper; certain features, however, may be referred to.

Station Yard Buildings.—The Station yard buildings are small, and are at convenient points in the yard to provide offices, store-rooms, locker- and toilet-rooms, for the use of employees on duty at the track level. All are of fire-proof construction, and of uniform architectural design; with the exception of Nos. 2 and 5 (which are of hollow tile covered

TABLE 6.—(*Continued.*)

Description.	Location.	Type of construction.	Size, in feet.
STATION YARD—(*Continued*).			
U. S. Post Office	Eighth Avenue, 31st to 33d Streets	Granite	335 by 375
Yardmaster's Office. One story	Eighth Avenue and 33d Street	Reinforced concrete	13 by 38
Trackmen's Building. One story	Eighth Avenue and 33d Street	Tile and stucco	9 by 19
Signal Cabin "B." One story and basement	Eighth Avenue and 33d Street	Reinforced concrete	10 by 36
Track, Signal, and Third-rail Tool-house. One story.	33d Street between Seventh and Eighth Avenues	Expanded metal and stucco	21 by 86
Service Power-plant. Contains Sub-station No. 2	31st Street between Seventh and Eighth Avenues	Granite	97 by 161
Pennsylvania Station	Seventh to Eighth Avenues, 31st to 33d Streets	Granite	430 by 784
34th Street. Exit Stairway, Head-house	33d Street between Seventh and Eighth Avenues	Brick	30 by 92
Assistant Station Master's Office. One story	Seventh Avenue and 33d Street	Tile and stucco	9 by 12
Assistant Yard Master's Office. One story	Seventh Avenue and 32d Street	Reinforced concrete	10 by 36
Signal Cabin "C." One story and basement	Seventh Avenue and 33d Street	Reinforced concrete	12 by 35
Signal Cabin "D." One story and basement	Seventh Avenue and 32d Street	Reinforced concrete	11 by 35
EAST RIVER SECTION			
Shaft-house, Tunnels 4 and 3. Auxiliary Sub-Station "D"	First Avenue, Manhattan	Brick	40 by 45
Shaft-house, Tunnels 2 and 1. Auxiliary Sub-Station "C"	First Avenue, Manhattan	Brick	40 by 45
Shaft-House, Tunnels 4 and 3	Front Street, Long Island City	Brick	40 by 64
Shaft-house, Tunnels 2 and 1. Auxiliary Sub-station "B"	Front Street, Long Island City	Brick	40 by 64
Traction Power-house. Sub-Station No. 1	Front Street, Long Island City	Brick	200 by 265
Trackmen's Tool-house	West of Portals 3 and 4, Long Island City	Frame	16 by 20
SUNNYSIDE YARD			
Telephone Terminal House	Over Portals 1 and 3	Frame-stucco	8 by 9
Signal Cabin "F." Two stories and basement	West of Thompson Avenue	Brick	17 by 27
Trackmen's Tool-house. One story	West of Thompson Avenue	Frame	16 by 20
Signal Cabin "Q," Yardmaster's Office. Two stories.	East of Queensboro Bridge Approach	Brick	14 by 83
Sand-house. One story	160 ft. east of Queensboro Bridge Approach	Brick	25 by 32

TABLE 6.—TERMINAL RAILROAD BUILDINGS; LISTED CONSECUTIVELY FROM THE WEST.

Description.	Location.	Type of construction.	Size, in feet.
MANHATTAN TRANSFER YARD.			
Signal Cabin "N" (Switch Station No. 4-B, in basement)	West of Transfer Station	Brick	17 by 27
Signal Cabin "S"	East of Transfer Station	Brick	17 by 27
Trackmen's Tool-house	East of Transfer Station	Frame	16 by 20
Transfer Station and Yard Buildings	In Transfer Yard	Described in paper by E. B. Temple, M. Am. Soc. C. E., *Transactions*, Vol. LXVIII, p. 75.	
MEADOWS SECTION.			
Trackmen's Tool-house	On Bridge over N. Y. Div.	Frame	16 by 20
Sub-Station No. 4	East of Bridge over N. Y. Div	Brick	52 by 105
Switch Station No. 4-A	Hackensack River Bridge	Galvanized iron	9 by 13
Signal Cabin "W"	Hackensack River Bridge	Galvanized iron	9 by 13
Switch Station No. 3-B	West of Erie Railroad Yard	Brick	9 by 11
Trackmen's and Third-rail Tool-house. Triple House.	Hackensack Portals	Frame	20 by 47
Sub-Station No. 3	Hackensack Portals	Brick	52 by 105
NORTH RIVER SECTION.			
Blower-House	Over Hackensack Portals	Granite	24 by 51
Employees' Dwellings. Two double two-story houses.	Above Hackensack Portals	Brick	31 by 46
Trackmen's Tool-house	Weehawken Shaft	Expanded metal and stucco	7 by 16
Shaft Head-house	Eleventh Avenue Shaft	Corrugated iron	27 by 39
STATION YARD.			
Track and Signal Building	Tenth Avenue Portals	Reinforced concrete	12 by 25
Car Battery-Charging House. Two stories	31st Street and Ninth Avenue	Reinforced concrete	12 by 19
Car Inspectors' and Cleaners' Building. Two stories.	Ninth Avenue and 33d Street	Reinforced concrete	10 by 49
Signal Cabin "A"	Between Ninth Avenue and Post Office	Steel-concrete	21 by 62

TABLE 6.—(*Concluded.*)

Description.	Location.	Type of construction.	Size, in feet.
SUNNYSIDE YARD—(*Continued*).			
Carpet shed	50 ft. east of Sand-house	Steel shed	34 by 89
Commissary Building. P. R. R.—67 by 97 ft. Pullman—67 by 161 ft. Two stories	100 ft. east of Carpet shed	Brick	67 by 258
Hose-house	South Yard, west of Honeywell Street	Frame	6 by 6
Store-house. Two stories	56 ft. east of Commissary building	Brick	67 by 162
Car-Battery house. One story	30 ft. east of Store-house	Brick	67 by 103
Pipe-rack	30 ft. east of Car-Battery house	Corrugated iron	21 by 21
Aux. Sub-Station "A" and Boiler-house. One story	181 ft. east of Pipe-rack	Brick	51 by 253
Hard coal and charcoal bin	25 ft. east of Boiler-house and 35 ft. west of Honeywell Street	Frame	11 by 81
Switching Station No. 1-A. One story	South of South Yard, West of Honeywell Street	Brick	18 by 35
Tool-house	65 ft. east of Honeywell Street	Frame	16 by 20
Oil-house	250 ft. east of Honeywell Street	Brick	51 by 67
Engine-house	60 ft. east of Oil-house	Concrete	72 by 161
Scrap-bin	79 ft. east of Engine-house	Frame	12 by 36
Iron-rack	29 ft. east of Scrap-bin	Corrugated iron	21 by 21
Signal Cabin "H." Two stories and basement	600 ft. west of Harold Avenue	Brick	17 by 27
Wheel-shop. One story	50 ft. east of Iron-rack	Expanded metal stucco	41 by 81
Auxiliary Store-house	Between Wheel-shop and Wheel-shed	Expanded metal stucco	10 by 30
Wheel-shed	60 ft. east of Wheel-shop	Steel shed	50 by 100
Signal Cabin "R." Two stories and basement	200 ft. west of Harold Avenue	Brick	17 by 27
Employees' Dwellings. Two double, two-story houses	Laurel Hill Avenue	Brick	31 by 46

FIG. 1.—INTERIOR OF INTERLOCKING CABIN "A."

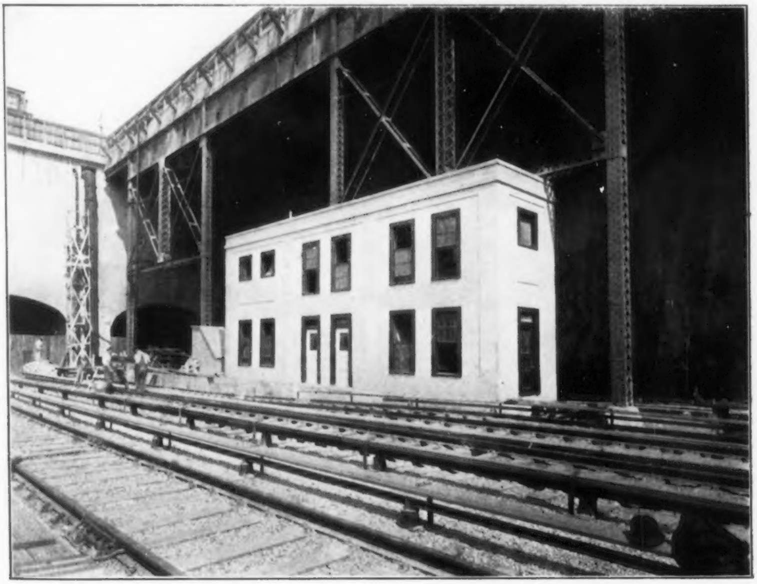


FIG. 2.—CAR-CLEANERS BUILDING, EAST OF NINTH AVENUE.

with cement plaster), they are of reinforced concrete without exterior ornamentation. The doors and windows are set in depressed panels, and a parapet accented by raised band courses finishes the roof. It will be noted from Table 6 that nearly all buildings have toilet facilities, which, in fact, was an essential feature in determining their location, as sewer connections had to be made through ejectors located at fixed points in the subway system under the tracks.

Signal Cabins.—There are four signal cabins in the Station yard, and all are of special design to meet special physical and operating conditions. Three of the cabins are under the buildings and street viaducts. They are of restricted dimensions because of the close clearances, and of irregular shapes in order to obtain the maximum room inside and the least obstruction to the operator's view of the tracks. Cabin "A," Plate LXXXV, is the main interlocking station of the Terminal, controlling all the movements to and from the west and the main-ladder switching movements. In order to obtain a central location and unrestricted view, the building was placed on a bridge over the throat of the ladder tracks about midway between the Post Office Building and Ninth Avenue. This gives the cabin a prominent position in the open yard, and it was thought to justify the design of a somewhat pretentious structure. The building is of the monolithic concrete type, and perhaps might be termed a reinforced concrete building, although, because of its spanning the tracks, and the fact that it serves as a support for the overhead third-rail and signal structures, a considerable quantity of structural steel is buried in its walls; thus it is not a purely reinforced concrete structure. The architecture is of the Mission type, with wide overhanging eaves and low ridged roof, covered with red Spanish tile. Owing to its location over and adjoining the switchwork of the tracks, special care was taken to protect the supporting walls in case of train derailment. At the east the station platforms perform this function, and on the west were placed very massive wedge-shaped fenders composed of 80-lb. T-rails embedded in concrete and carried below and under the track system. The entire floor space of the cabin is occupied by the operating-room, containing the interlocking machine on the floor and the relays and wiring in a gallery above; the gallery girders also act as anchor arms for the cantilever structures attached to the cabin and used for supporting the overhead rail and signals.

All signal wiring is carried in a false floor above the concrete floor of the cabin and in ducts through the side-walls and into a basement extending under the entire cabin below the tracks, where switch-boards, storage batteries, and other apparatus are located. From this basement the wire conduits are carried in a special subway (referred to elsewhere) which communicates with the subways under the yard. Fig. 1, Plate LXXXVI, is a view of the interior of this cabin.

The remaining three cabins, "B," "C," and "D" are under other structures; they are of two stories, a main operating floor and a basement or track-level story. The buildings are of the usual reinforced concrete construction. Fig. 2, Plate LXXXVI, is a view of the car cleaners' building east of Ninth Avenue.

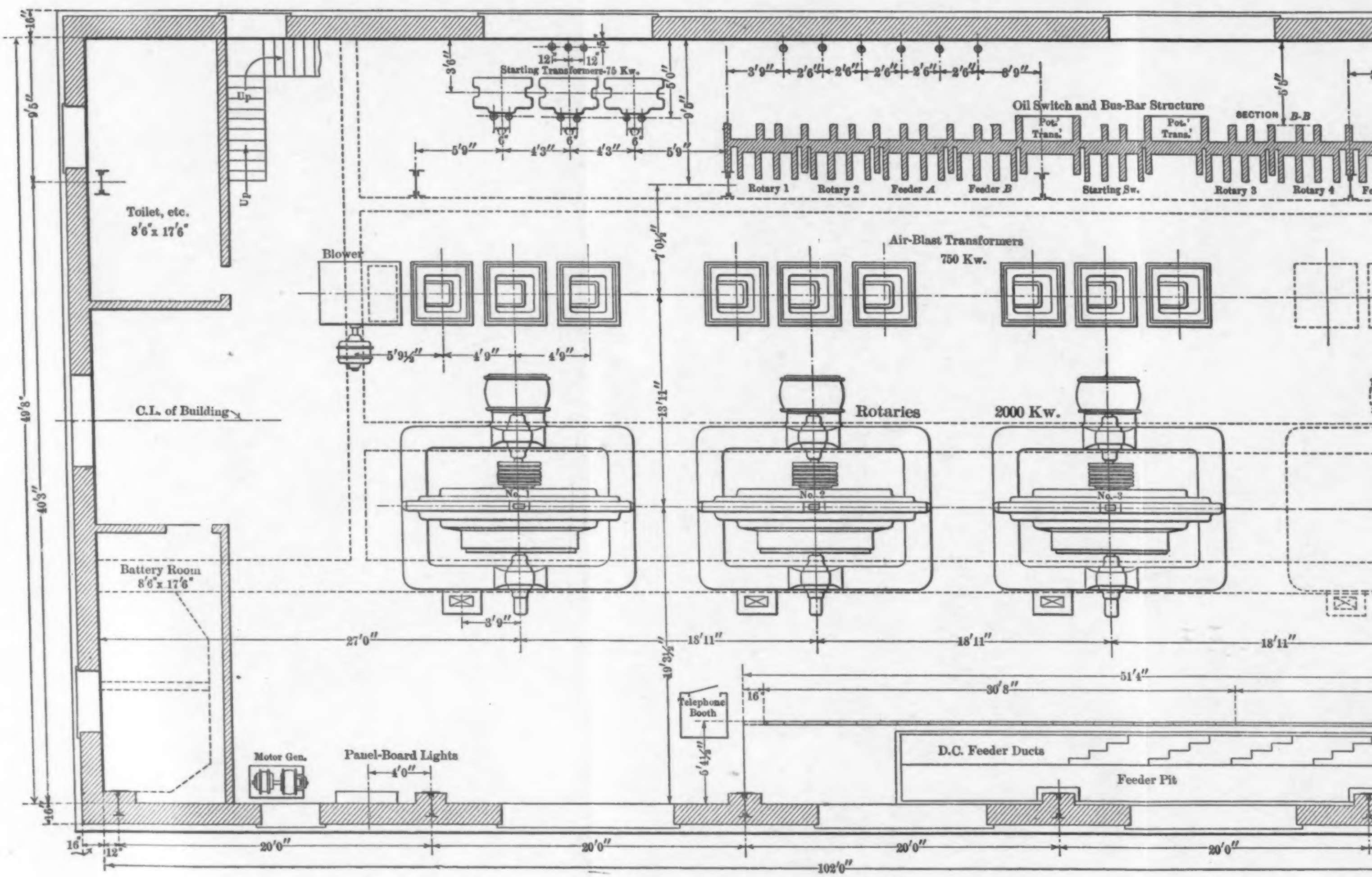
Manhattan Transfer.—Only signal cabins "N" and "S" were constructed by the writer's Department; the transfer station was designed and built by Mr. Shand as part of the general yard facilities. The signal cabins are of the usual standard design of the Pennsylvania Railroad Company.

Meadows Section.—Aside from the power sub-station buildings, these comprise small structures for electric power switching, tool-houses, and an interlocking cabin at the Hackensack draw-bridge. These latter have steel frames carried by brackets on the bridge approach, and are covered with paneled galvanized iron and lined with asbestos board.

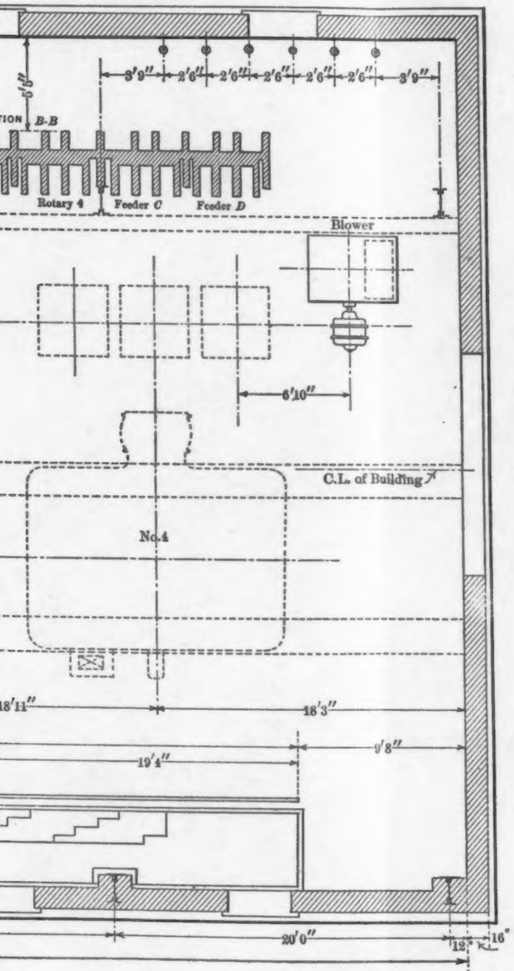
Sunnyside Yard.—The buildings in the Sunnyside yard (Plate LXXXVIII) are numerous and important, as is seen from Table 6, and are quite plain and of fire-proof construction. They adjoin the service yard, and are devoted to the various motive-power requirements. The outside and party walls are of hard-burned red brick, except in the cases of the engine-house and the wheeling shed, which are of steel frame covered with expanded metal and concrete plaster. The floors are generally of concrete and the roofs of steel covered with book-tile, roofing-felt, and gravel. The buildings are provided with steam heating, hot and cold water, toilet facilities, fire protection, electric lights and telephones. The interiors, with few exceptions, are fitted with metal shelving, bins, and lockers.

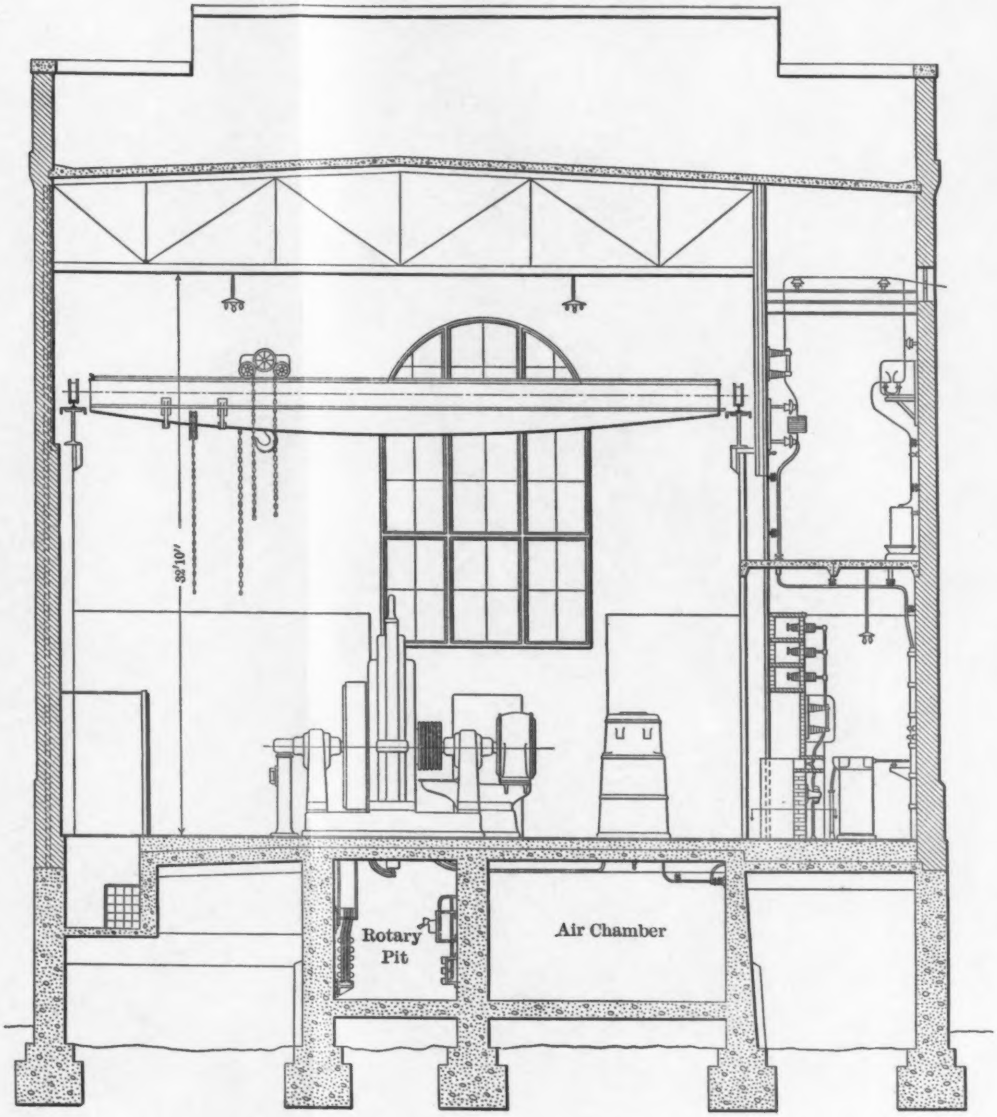
The engine-house and machine-shop are especially designed for ample light. The engine-house has two inspection pits, with a cross-

PLAN AND CROSS-SECTION OF I



TION OF HARRISON SUB-STATION





pit fitted with compressed-air jacks for removing wheels. The machine-shop contains motor-driven machine tools for light repairs.

The four signal cabins are of brick, and of standard Pennsylvania Railroad Company type, except Cabin "Q," which is special and has an extension containing tool-rooms, yardmaster's offices, etc.

Power Sub-stations.—Sub-stations Nos. 1 and 2 are in the Long Island City Power-House and in the 31st Street Service Plant, respectively; Nos. 3 and 4 are on the Meadows Division, No. 3 adjoining the Hackensack Portal and No. 4 at the east end of the Manhattan Transfer yard. These latter buildings are similar in design and construction, and entirely fire-proof. Their exteriors are of dark red mottled brick, laid with irregular bond, and surmounted by parapet walls; the roofs are of concrete, and flat; the interior in each case (see Plate LXXXVII) consists of a main operating-room with small partitioned rooms for control storage batteries, toilets, and lockers. The main room contains a gallery on one side for lightning arresters and remote-control circuit-breaker apparatus.

Blower-Houses.—Special buildings were required at certain locations for housing the tunnel-ventilation apparatus. At the Hackensack Portal a granite building was constructed for the purpose by the Chief Engineer of the North River Division. At the First Avenue Tunnel shafts two buildings were provided; one is founded on the shaft caissons, which are concrete-lined steel shells, and project slightly above the ground surface, and the other adjoins the second pair of caissons. These buildings are two stories high, approximately square, of dark red brick with irregular bond, and with concrete floors and roofs; they are thoroughly fire-proof. The interiors contain fans belted to induction-type electric motors; the ducts from the fans to the shafts are large concrete funnels with "ferro-inclave" reinforcement, carefully designed for leading the air to the shafts with minimum friction. The buildings also contain switching apparatus for the fan motors, for operating the sump pumps, and for controlling the tunnel lighting. At the Long Island City shafts, two buildings are provided for the same general purposes. These are shaped like the letter Z (see Fig. 10), and are placed over the caissons and between the shaft openings.

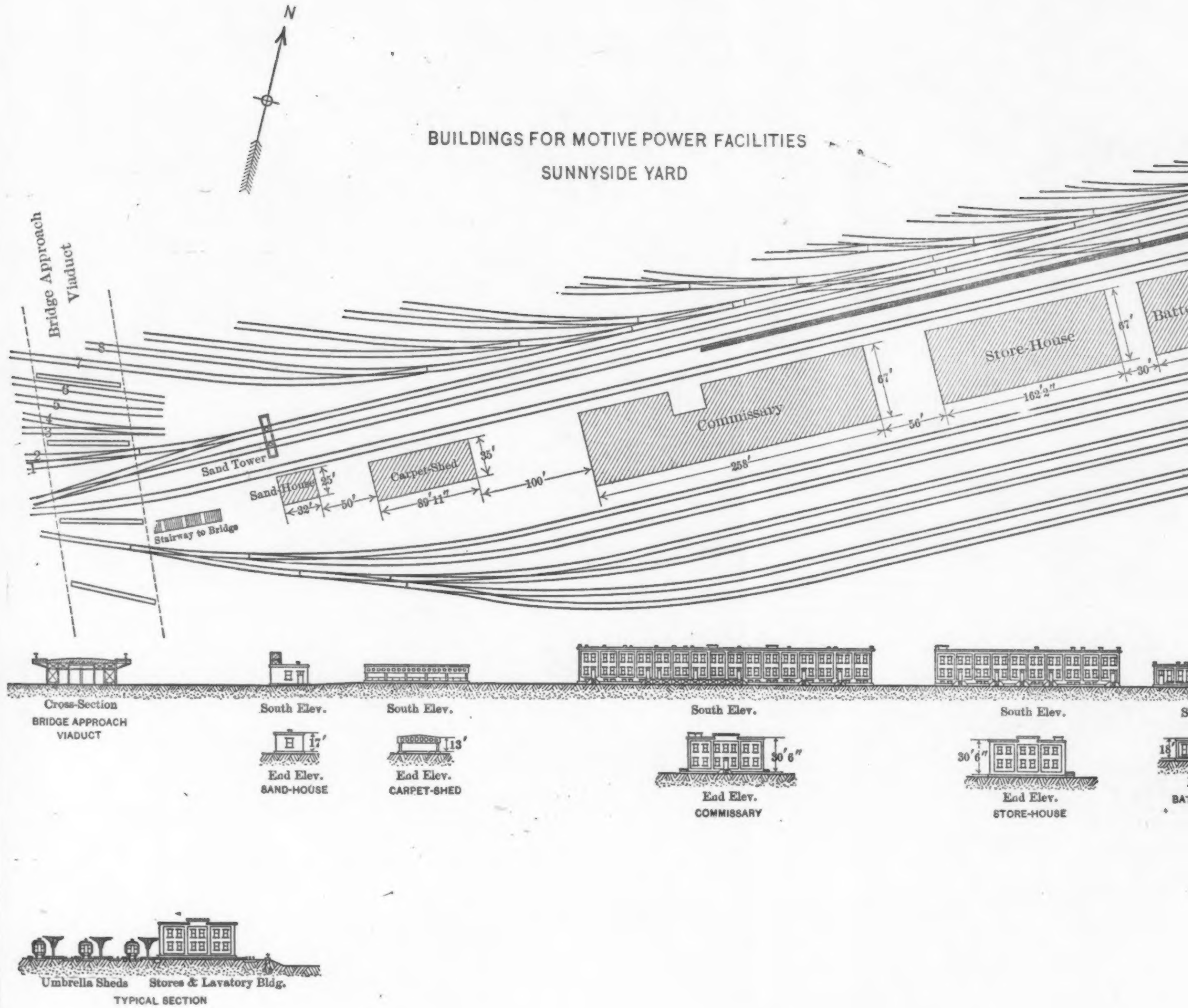
From each pair of tunnels three stairways rise to the surface and

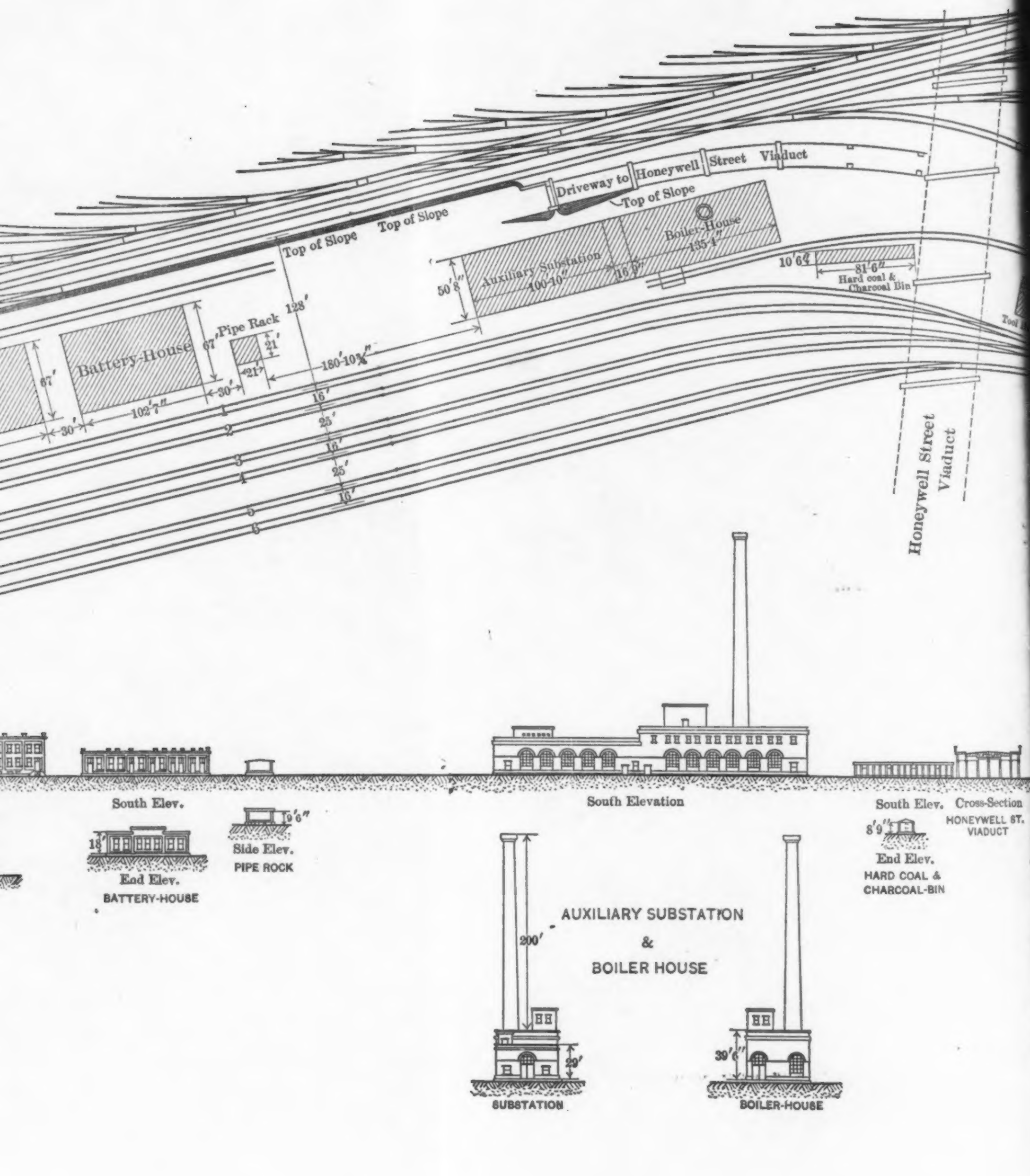
terminate in small reinforced concrete kiosks; the shaft openings, for lowering machinery, etc., are enclosed by circular concrete fences, having steel doors. Around each blower-house shaft opening a reinforced concrete wall 8 ft. 6 in. high has been built, enclosing permanently a plot of ground which it has been necessary to reserve.

Sunnyside Yard.

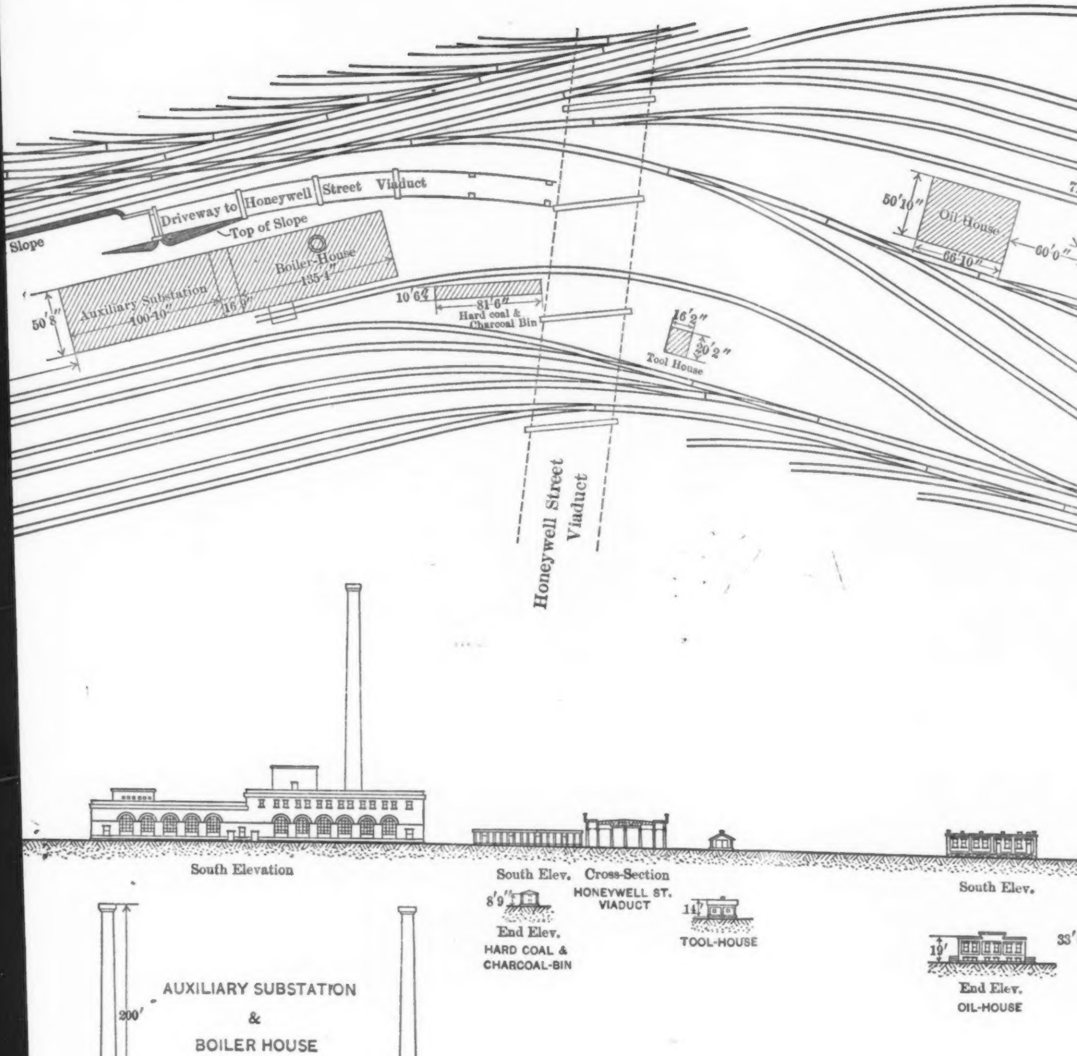
This extensive yard is at the east end of the Terminal Railroad, in Long Island City. Its purpose is to furnish facilities for the storage and care of passenger train equipment using the New York Station. Practically all long-distance trains arriving at the Pennsylvania Station, when unloaded, are taken to Sunnyside Yard for turning, cleaning, and making up for the return trip. At present the Long Island Railroad does not make use of the Sunnyside Yard facilities, the turning of trains being done in the main station yard. The track plan, Plate CV, was devised by Mr. L. H. Barker, Resident Engineer. While the yard is a stub-end one, as regards its location at the end of the Division, yet it is double-end as regards train movements; this is accomplished by providing two loop tracks from the tunnels around the yard to its further end. Trains arriving from the New York Terminal, therefore, may enter the yard in the reverse direction and be ready to return to the Station in the same head-end order, as generally required, thus minimizing the shifting and turning of special cars on a table. Furthermore, conflicting movements at the throat of the yard are avoided. It is important to note that the tunnel tracks from the New York Station are operated as two double-track lines, one (the 33d Street Tunnels) normally for Long Island Railroad trains, and the other (the 32d Street Tunnels) for movements to and from Sunnyside Yard. The Long Island Railroad trains, from the tunnels and from Long Island City, pass through the yard at a higher level than the yard tracks, and without grade crossings of any kind. In order, however, to give access to the yard from all tunnels and in the minimum distance, it was necessary to cross the west-bound 32d Street over the east-bound 33d Street Tunnel near the portals. Thus the yard may be entered at either end by the two east-bound tracks without grade crossings. A short distance east of the yard there is a jump-over connection and junction with the proposed New York Connecting Railway, a double-track line to connect

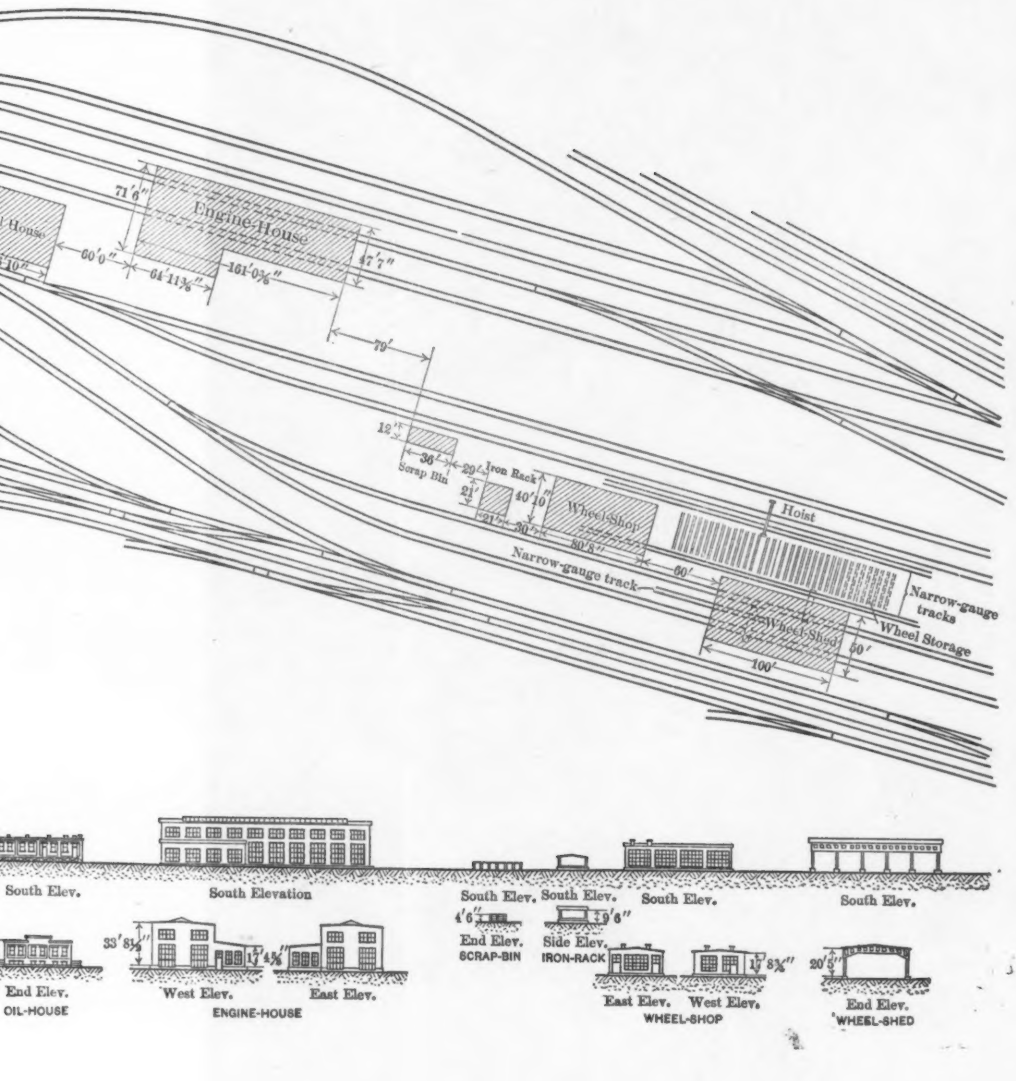
BUILDINGS FOR MOTIVE POWER FACILITIES
SUNNYSIDE YARD





Driveway to Honeywell Street Viaduct
Top of Slope
Top of Slope
Top of Slope
Auxiliary Substation 100'10"
Boiler-House 135'4"
16'9"
50'8"
10'6"
81'6"
Hard coal & Charcoal Bin
Battery-House
67'
67'
102'7"
30'
30'
Pipe Rack
21'
21'
128'
180'10"
16'
25'
16'
25'
16'
1
2
3
4
5
6
Honeywell Street Viaduct
South Elev.
End Elev.
BATTERY-HOUSE
18'
9'6"
Side Elev.
PIPE ROCK
South Elevation
AUXILIARY SUBSTATION
&
BOILER HOUSE
200'
29'
39'6"
SUBSTATION
BOILER-HOUSE
South Elev.
8'9"
End Elev.
HARD COAL & CHARCOAL-BIN
Cross-Section
HONEYWELL ST. VIADUCT


Slope
Driveway to Honeywell Street Viaduct
Top of Slope
Boiler-House
135'4"
Auxiliary Substation
100'10"
16'9"
50'8"
10'6"
81'6"
Hard coal & Charcoal Bin
16'2"
20'2"
Tool House
50'10"
Oil-House
66'10"
60'0"
Honeywell Street Viaduct
South Elevation
South Elev.
Cross-Section
HONEYWELL ST. VIADUCT
8'9"
End Elev.
HARD COAL & CHARCOAL-BIN
14'
TOOL-HOUSE
South Elev.
19'
33'
End Elev.
OIL-HOUSE
AUXILIARY SUBSTATION
&
BOILER HOUSE
200'
29'
SUBSTATION
39'6"
BOILER-HOUSE


Engine-House
71'6"
60'0"
64'11⅜"
161'0⅜"
47'7"
79'
12'
36'
Scrap Bin
29'
Iron Rack
21'
40'10"
30'
Wheel-Shop
80'8"
Hoist
Narrow-gauge track
60'
Narrow-gauge tracks
Wheel-Shed
Wheel Storage
50'
100'
South Elev.
South Elevation
South Elev.
South Elev.
South Elev.
South Elev.
4'6"
9'6"
End Elev.
SCRAP-BIN
Side Elev.
IRON-RACK
33'8½"
17'4⅜"
17'8⅜"
20'5"
End Elev.
OIL-HOUSE
West Elev.
East Elev.
ENGINE-HOUSE
East Elev.
West Elev.
WHEEL-SHOP
End Elev.
WHEEL-SHED

with the New York, New Haven and Hartford Railroad by a bridge over Hell Gate. The main freight connections of the Long Island Railroad to Long Island City pass around the yard to the north, and cross the yard approach by overhead bridges near the tunnel portals.

The main, or south, yard has an ultimate capacity for 861 cars, and the supplemental, or north, yard has a capacity of 526 cars; the present capacity, number of tracks, etc., are shown by the summary of statistics on page 732. The service buildings are between the north and south yards, as shown on Plate LXXXVIII. Fig. 1, Plate LXXXIX, is a view of the south yard looking eastward from Honeywell Street Viaduct; and Fig. 2, Plate LXXXIX, is a view of the Sunnyside Yard buildings looking southeastward from the north yard.

The south yard, which is used for cleaning and making up trains, is provided with platforms between tracks for trucking purposes, and a complete piping system for air, water, and steam, as well as conduits and wiring for charging train-lighting batteries, all having connections for each car on each track. All tracks in both north and south yards are equipped with third-rail, so that electric motive power is available throughout for shifting trains. Fig. 1, Plate XC, is a view of the main tracks of the Long Island Railroad looking eastward from Thompson Avenue Viaduct, and showing the entrance to the yard. Fig. 2, Plate XC, is a view of Sunnyside Yard looking eastward from the tracks at the west end of the south yard.

Piping.—The piping and wiring systems are installed in a permanent and conveniently accessible manner. Thus, from the boiler-house and auxiliary power sub-station a cross pipe-tunnel 603 ft. long has been run at right angles to the main yard tracks, with openings to the inter-track spaces (see Fig. 11). Branching from this tunnel are concrete trenches, one between each alternate track, running the entire length of the tracks. In the walls of the tunnel and trenches are installed conduits for the battery-charging wires, the system being centrally operated from the sub-station; the tunnels and trenches also contain pipes for air, water, and steam. The trenches provide drainage for surface-water, and connect to the yard sewer system at suitable points.

Water Supply.—A complete local water supply is derived from wells sunk within the yard area, furnishing sufficient water for all the requirements of the yard, up to 1 000 000 gal. per day, and, in

addition, the requirements of the Long Island City power-house, the total being about 2 000 000 gal. per day. There are two wells, each about 30 ft. deep. They are operated by direct suction through pipes to the pumps in the basement of the boiler-house. The pumps supply the yard service piping system direct and, by a main line through the yard, a 250 000-gal. storage tank at Borden Avenue, near the tunnel portals, and thence to the Long Island City power-house.

Yard Lighting.—The general illumination of the yard is effected by forty-one 3 000-c.p. flaming-arc lamps of the long-burning type. These are mounted on steel poles from 200 to 350 ft. apart, having special reference to important local points. Direct current at 110 volts is supplied to the lamps from the battery-charging motor-generator sets in the auxiliary sub-station. The interior of the auxiliary power sub-station is shown on Fig. 1, Plate XCI, and Fig. 2, Plate XCI, shows the switch-board.

Yard Buildings.—The yard buildings have the following functions:

(*a*) A sand-house, equipped with sand-drying stoves, storage, and pneumatic elevating machinery for sanding locomotives.

Equipment:

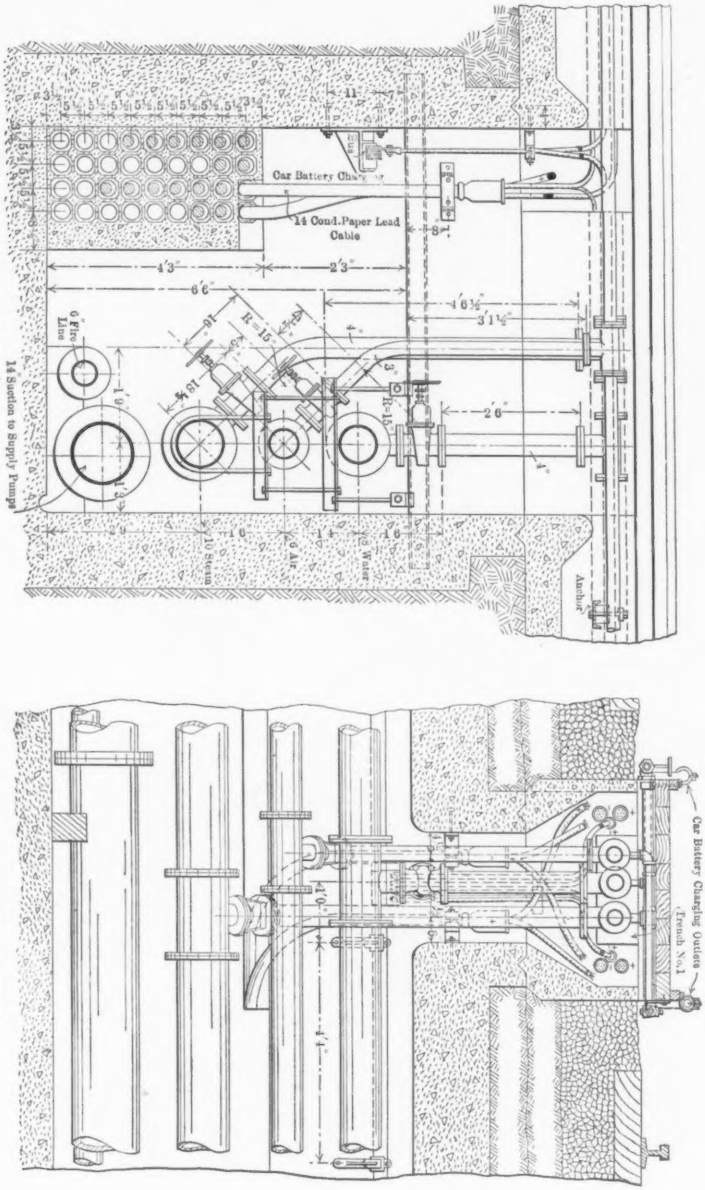
Concrete bins for the storage of 2 000 cu. ft. of wet sand and 550 cu. ft. of coal.
Two Pennsylvania Railroad standard sand dryers.
Two pneumatic dry sand tanks, 35 cu. ft. each,
Sand and air piping complete, and connections, with
Two gravity locomotive sanding tanks, 35 cu. ft. each.

(*b*) A carpet shed, equipped with benches and air cleaners for carpets.

Equipment:

Twelve carpet racks, each 12 by 12½ ft. and 2 ft. high, fitted with compressed air and vacuum nozzles for cleaning carpets.

(*c*) A commissary building containing separate provisions for the Railroad and the Pullman Company, for the storage of supplies for dining and sleeping cars, including a motor-driven refrigerating plant with cold boxes for perishable supplies. Also a bunk-room for Pullman employees.



TYPICAL SECTIONS OF MAIN PIPE TUNNEL, SUNNYSIDE YARD.

Fig. 11.

Equipment of P. R. R. Section:

Ten refrigerator boxes, ranging in capacity from 90 to 1 824 cu. ft., each equipped with direct-expansion ammonia coils for keeping meat, fish, poultry, fruit, ice cream, cheese, milk, butter, etc.
Two motor-driven refrigerating machines, 4 tons each.
One 3 000-lb. direct-connected electric elevator.
One dumb-waiter.
Two 250-gal. water filters, each with two copper reservoirs for storing filtered water.
One two-oven range with heater, steam table, and kitchen equipment complete.
Three separate toilet-rooms, with hot and cold water and fixtures.
Four hose connections for fire, and 75 ft. of 2½-in. hose, with nozzles.
General commissary storage-rooms, wine-room, kitchen, silver-room, record storage room.
Clean- and soiled-linen room, linen-repair room.
Superintendent's and clerks' offices, kitchen and porters' room.
Conductors' room, waiters' and cooks' room.
All rooms furnished with metal shelving and lockers.
Freight, team, and distributing platforms at each side of building.

Equipment of Pullman Section:

One general refrigerator box, about 500 cu. ft., with direct-expansion ammonia coils, and congealing tanks.
One motor-driven refrigerating machine, ½ ton capacity.
One 3 000-lb. direct-connected electric elevator.
Three separate toilet-rooms, with hot and cold water and fixtures.
Four hose connections for fire, and 75 ft. of 2½-in. hose with nozzles.
General storage-rooms, soiled- and clean-linen rooms.
Linen-repair room, with air and electric sewing machines.
Carpet- and carpet-repair-rooms.
Commissary-rooms, plumbers', carpenters', and testing-rooms.
Superintendent's, storekeeper's and foreman's rooms.
Porters' and cleaners' quarters and sleeping-rooms.

(*d*) A general store-house for the Railroad Company's stores (except oil), required for repairs in the yard and cars; also lavatories and closets for employees.

FIG. 1.—SUNNYSIDE YARD: VIEW OF SOUTH YARD, LOOKING EASTWARD FROM HONEYWELL STREET VIADUCT.

FIG. 2.—SUNNYSIDE YARD BUILDINGS: LOOKING SOUTHEASTWARD FROM NORTH YARD.

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ro
ap

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Equipment:

One 5 000-lb. direct-connected electric elevator.
One 5 000-lb. P. R. R. dormant store-room scale, and portable store-room scales.
One stationary triple-valve test set, with four portable test sets complete.
Three separate toilet-rooms with hot and cold water and fixtures.
Foreman's, storekeeper's and clerks' offices.
General store-rooms, carpet-rooms.
Air brake and pipe fitters', tin and carpenter shops.
Car cleaners' quarters and locker-rooms for both men and women.
All rooms furnished with metal shelving and lockers.
Three hose connections for fire, with 50 ft. of $2\frac{1}{2}$-in. hose and nozzles.
Freight, team, and distributing platforms each side of building.

(*e*) A battery-repair house for car-lighting batteries, with a separate room for charging batteries and for spare batteries; also a distilling apparatus to supply pure water for refilling batteries.

Equipment:

Three 14-circuit switch-board panels, for battery charging.
One $17\frac{1}{2}$-gal. per hour water still and reservoir.
One battery plate press and battery tools.
One motor-driven emery grinder and buffer.
One motor-driven sensitive drill press.
Two 1-ton chain falls and trolleys.
Two lead-covered battery cleaning benches with hot and cold water.
Battery assembling and charging benches.
Acid mixing and storage tanks, lead-lined.
Train lighting and repair-room, with benches and tools.
Lamp-frosting room, and general store-room.
Oxygen and hydrogen tanks.

(*f*) A boiler-house and auxiliary power sub-station (shown on the upper part of Plate LXXXIII), containing boilers with feed pumps, water heaters, etc., for supplying steam for all purposes in the yard; also pumps for water supply system and for fire service, air compressors for the air supply to the yard and to the signal system, and the auxiliary sub-station apparatus, as described under the heading "Auxiliary Sub-station." Fig. 12 is a cross-section of the boiler-house.

Equipment of Auxiliary Sub-station Section:

One 10-ton hand-power crane.
Two 1 500-cu. ft., two-stage, steam-driven, air compressors, with inter-coolers, reservoirs, traps, water-cooled after-cooler, and atmospheric after-cooler.
Office, toilet, and locker-rooms, and battery-room.
Three 250-kw., 110-220-volt, 3-wire, motor-generator sets for car battery charging, building lighting, and power.
Nine 125-kw., air-blast transformers.
Two 10 000-cu. ft. blowers for cooling transformers.
Thirty-four switch-board panels; 52 ultimately.
Eighteen car battery charging rheostats; 34 ultimately.

Equipment of Boiler-House Section:

Three 500-h.p. water-tube boilers, with chain-grate stokers.
One 20-ton, motor-driven, coal and ash-handling conveyor.
Four coal bunkers, 30 tons each.
One 20-ton motor-driven, coal crusher.
One steel, brick-lined, smoke-stack, 200 ft. high, 9 ft. in diameter at top.
Steel smoke flues, lined with reinforced magnesia and retort cement.
One 1 500-gal., boiler feed-water heater.
Two 150-gal., duplex, boiler feed pumps.
Soda ash system for treating boiler water.
Three 500-gal., compound, duplex-service water and fire pumps.
One 7½-h.p. steam engine, and one 10-h.p. motor for driving stoker.
One steam whistle, for fire protection.
One 40-gal. Chemical engine.
One hose cart, and 1 000 ft. of 2½-in. hose.

(*g*) An oil-house, containing tanks for storage of the various kinds of oil needed for lubrication of cars and machinery, with measuring, cleaning, and pumping apparatus.

Equipment:

Complete metal equipment for waste, paints, and lamps.
Ten long-distance, self-measuring pumps.
Eleven short-stroke oil pumps.
Twenty-one oil-storage tanks.
One oil filter.
Two sponging tanks.
Complete oil-handling system.

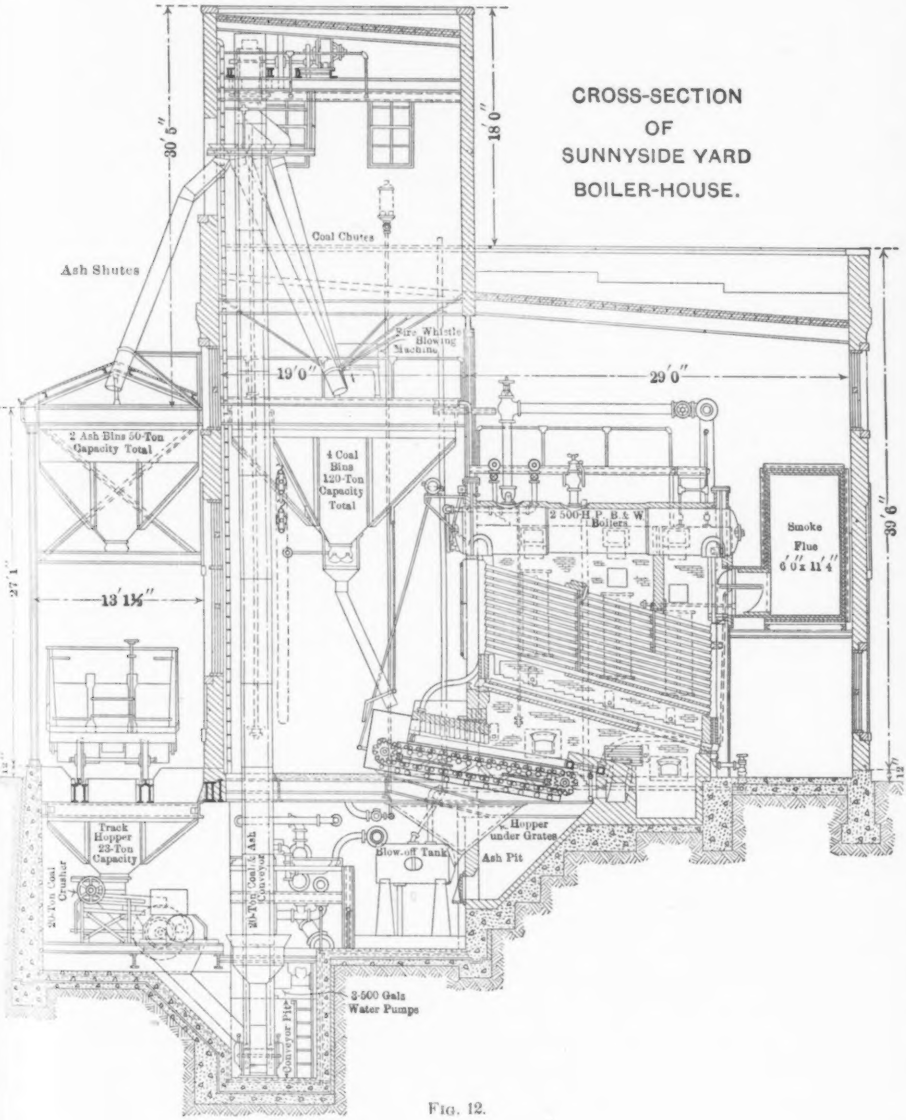
CROSS-SECTION
OF
SUNNYSIDE YARD
BOILER-HOUSE.
30′ 5″
18′ 0″
Coal Chutes
Ash Shutes
Fire Whistle Blowing Machine
19′ 0″
29′ 0″
2 Ash Bins 50-Ton Capacity Total
4 Coal Bins 120-Ton Capacity Total
2 500-H. P. B. & W. Boilers
Smoke Flue 6′ 0″ x 11′ 4″
39′ 6″
27′ 1″
13′ 1½″
12″
Track Hopper 23-Ton Capacity
20-Ton Coal Crusher
20-Ton Coal & Ash Conveyor
Blow-off Tank
Hopper under Grates
Ash Pit
2-500 Gals Water Pumps
Conveyor Pit

FIG. 12.

(*h*) An engine-house, with machine-shop, for housing electric locomotives during inspection. It is equipped with an electric traveling crane, pit jacks for dropping driving wheels, and machine tools for light repairs. Heavy locomotive repairs and general overhauling are to be done at the Meadows Shops of the Company.

Equipment:

Offices, lockers and toilet-rooms with fixtures.
Two inspection pits.
One wheel pit, with pneumatic wheel jack and two air reservoirs.
One 25-ton, three-motor, electric, traveling crane.
Three lathes, one shaper, one boring machine, one framer, one rod and box press, one radial drill, one 2-spindle drill press, one power hack-saw, one grindstone, two emery grinders, one 2-ton crane and trolley, and all small tools.

(*i*) A wheel shed, provided with pits and air lifts for changing wheels on coaches, with a wheel-storage yard adjoining, equipped with an electric hoist.

Equipment:

Two wheel pits, with two pneumatic wheel jacks and two air tanks.
Two inspection pits.
One 2-ton, electric hoist, trucks and track facilities, for handling wheels.

Yard Statistics.—The following additional statistics relate to the physical characteristics, operating quantities, etc., at Sunnyside Yard:

Extreme length of yard	8 815 ft.	
Extreme width of yard	1 625 "	
Area covered (3d Street to Laurel Hill Avenue)	192 acres.	
Present length of yard tracks (excluding main tracks)	25.72 miles.	
Length of main tracks in yard	11.09 "	
Ultimate length of yard tracks (excluding main tracks)	45.47 "	
	Present.	Ultimate.
Parallel tracks in south yard	34	77
" " " north "	12	42
Distance from Pennsylvania Station to east entrance of yard, *via* loop tracks	4.4 miles.	
Distance from west exit of yard to Pennsylvania Station	3.3 "	

	Present.	Ultimate.
Number of cars cared for daily in yard, for 1911 summer schedule	400	
Estimated ultimate daily capacity of yards, in cars cared for	1 000	
Standing room for cars in yard	503	1 387
Number of street viaducts over yard	5	
Total length of street viaducts	4 606 ft.	
Number of yard buildings	22	
Total length of between-track platforms	25 314 lin. ft.	
" " " umbrella sheds over platforms	2 798 " "	
Number of outlets at track for supplying cars with steam, water, air and electric battery charging	800	

Miscellaneous General Facilities.

The following operating facilities apply to the Division as a whole, and may best be referred to under a general heading.

Drainage.—The disposal of the leakage and storm-water from the Station yard and the extensive system of tunnels presented an important problem. The conditions to be met are:

(*a*) The disposal of storm-water in the open portion of the yard, and at the tunnel portals;

(*b*) Provision against possible breakage in water mains or sewers, which might cause flooding of the tunnels and yards;

(*c*) The disposal of seepage from beneath the yard retaining walls, and of leakage through the land and river tunnel linings, and drainage into the portions of the land tunnels which were built without iron lining, through pipes inserted for that purpose.

In estimating storm-water, a rainfall which would produce a run-off of 3 in. per hour was taken as the maximum. In the yard the estimate of seepage was based on experience under similar conditions; in the case of the tunnels, actual figures were obtained after the completion of the tunnel system. The likelihood of breakage in water mains or sewers in the streets around the yard and at the tunnel portals was carefully weighed, having regard to local conditions, and it was concluded that, given a reasonable capacity in the various sumps and pumps, which are necesary to care for the disposal of seepage and storm-water in the different sections of the tunnels, there would be sufficient reserve to care for any probable emergency conditions result-

ing from pipe breakages prior to the time when the water could be turned off. Compared with the possible quantity of storm-water, it has been found that the seepage to be cared for is quite small. The average seepage for each of the different sections, at the present time, is given in Table 7; these averages, however, may vary somewhat.

TABLE 7.—TUNNEL AND YARD SEEPAGE.

	Total gallons per minute.	Gallons per foot of tunnel per 24 hours.
EAST RIVER SECTION:		
Long Island City Shaft, Tunnels 1 and 2	23	4.2
" " " " " 3 " 4	52	9.7
River Tunnels, Lines 1 and 2	0.93	0.17
" " " 3 " 4	0.42	0.7
CROSS-TOWN SECTION:		
First Avenue Shaft, Lines 1 and 2	52	6.7
" " " " 3 " 4	51	6.6
STATION YARD AREA:		
Seventh Avenue Sump	40	......
Service Plant Sump	15	......
Ninth Avenue Sump	80	......
Tenth Avenue Sump	40	......
Eleventh Avenue Shaft	20	......
NORTH RIVER SECTION:		
River Tunnels (2)	0.8	0.085
Bergen Hill Tunnels (2)	70	8.428

NOTE.—In this paper the East River Tunnels are designated by numbers, instead of by the letters used in other papers; thus, as now established, Tunnel "D" becomes "1," "C" becomes "2," "B" becomes "3," and "A" becomes "4."

Pumping.—Seventeen pump sumps are provided for the system, from Hackensack Portal to Sunnyside Yard; their capacities and locations are given in Table 8. Fig. 2, Plate LXXXII, is a view of the interior of the Ninth Avenue sump, showing the vertical-shaft motors for the pumps. All pumps, except those at the lowest point in each of the six subaqueous tunnels, are of the submerged centrifugal type, primed at all times (see Fig. 13); they are set horizontally, and driven by vertical shafts from electric motors located in rooms above the sumps. The motors operate by three-phase, alternating, 60-cycle current at 440 volts, supplied, through transformers from the 11 000-volt circuits for lighting and auxiliary power, from the Service Plant. Each motor is controlled by a separate float-switch which starts and stops the pumps at certain pre-determined levels of water in the sump. Under normal conditions, only one sump pump operates, but the float-switch of the second pump is set so that the second pump will start also, in case the quantity of incoming water is so great that the level keeps

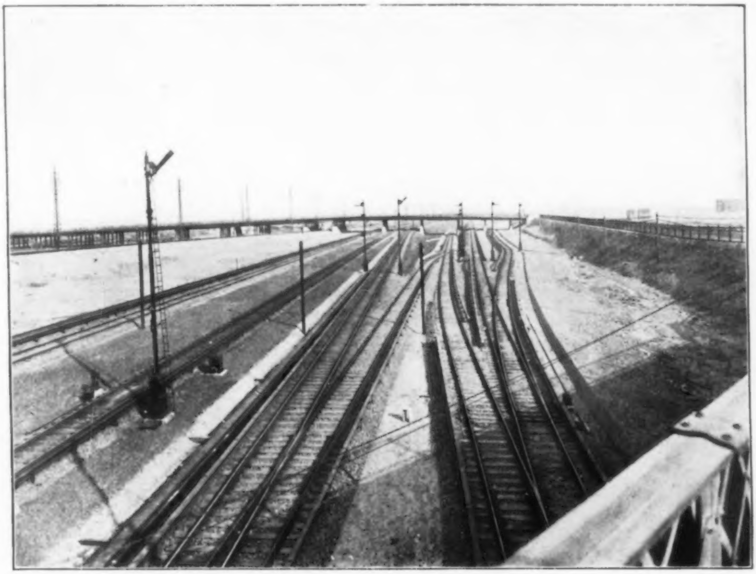

FIG. 1.—MAIN AND LOOP TRACKS, L. I. R. R., SUNNYSIDE YARD, LOOKING EASTWARD FROM THOMPSON AVENUE VIADUCT.

FIG. 2.—SUNNYSIDE YARD: LOOKING EASTWARD FROM TRACKS OF L. I. R. R. AT WEST END OF SOUTH YARD.

rising after the first pump is started. The floats being adjustable, the starting levels of the pumps are interchanged at certain intervals of time to insure all pumps being in proper operating condition. The air-driven pumps in the subaqueous tunnels have similar arrangements, except that the float operates a special air-valve instead of an electric switch.

TABLE 8.—DRAINAGE AND PUMPING SYSTEM.

Sump No.	Location.	Capacity of sump, in gallons.	Number of pumps.	Capacity of pumps, in gallons per minute.	Horse-power of motors.	Speed, in revolutions per minute.
1..	Sunnyside Portal, Tunnel No. 4....	26 000	1	500	7.5	1 120
2..	Sunnyside Portal, Tunnel No. 2.....	13 000	1	250	5.0	1 120
3..	Sunnyside Portal, Tunnels Nos. 1 and 3..............................	58 000	2	500	10.0	1 120
4..	Long Island City Shaft, Tunnels Nos. 3 and 4......................	59 000	1 1	100 1 000	7.5 40.0	1 700 1 120
5..	Long Island City Shaft, Tunnels Nos. 1 and 2......................	59 000	1 1	100 500	7.5 20.0	1 700 1 700
6..	1 000 ft. east of First Avenue Shaft, between Tunnels Nos. 3 and 4.....	16 000	2	100	$6\frac{1}{8}$ by 5 by 10 in.	42
7..	1 000 ft. east of First Avenue Shaft, between Tunnels Nos. 1 and 2.....	16 000	2	100	$6\frac{1}{8}$ by 5 by 10 in.	42
8..	First Avenue Shaft, Tunnels Nos. 3 and 4............................	54 000	2	100	7.5	1 700
9..	First Avenue Shaft, Tunnels Nos. 1 and 2............................	54 000	2	200	10.0	1 700
10..	Pennsylvania Station, Under tracks east of Platform No. 2...........	31 000	2	2 500	75.0	690
11..	Under 31st Street, Pennsylvania Station Service Plant.............	20 000	2	500	15.0	850
12..	Under Yard "C" at Ninth Avenue, Pennsylvania Station.............	100 000	2 1	1 250 250	240.0 15.0	580 1 700
13..	South Side, Tenth Avenue Portal...	37 000	2 1	4 100 250	100.0 15.0	690 1 700
14..	Eleventh Avenue Shaft.............	10 000 38 000	1 1	1 000 100	30.0 7.5	1 120 1 700
15..	2 200 ft. east of Weehawken Shaft, in west-bound tunnel.............	500	1 1	250 50	$4\frac{1}{2}$ by 4 by 8 in. 10 by 8 by 12 in.	50 40
16..	2 200 ft. east of Weehawken Shaft, in east-bound tunnel..............	500	1 1	250 50	$4\frac{1}{2}$ by 4 by 8 in. 10 by 8 by 12 in.	50 40
17..	Weehawken Shaft..................	37 000	2	250	15.0	1 700

The discharge from the centrifugal pumps is piped to the street surface and thence to the nearest sewer. The discharge pipe from the air-driven pumps is carried on the tunnel bench to the nearest shaft; thence to the surface, and to the nearest sewer.

The pumps in the subaqueous tunnels are of the duplex, air-driven piston type, operated from the compressed-air mains in the tunnels. This type was adopted, rather than the electrically-driven type, to insure operation in case the pump chamber became flooded. They are of small capacity only, as the quantity of water to be handled is not great.

Fire Protection in Station Yard.—The large open area of the Station yard and the superior fire-proof construction of the buildings furnish a very important fire barrier in the heart of New York City. This break extends from Seventh to Tenth Avenues, a distance of 2 800 ft.

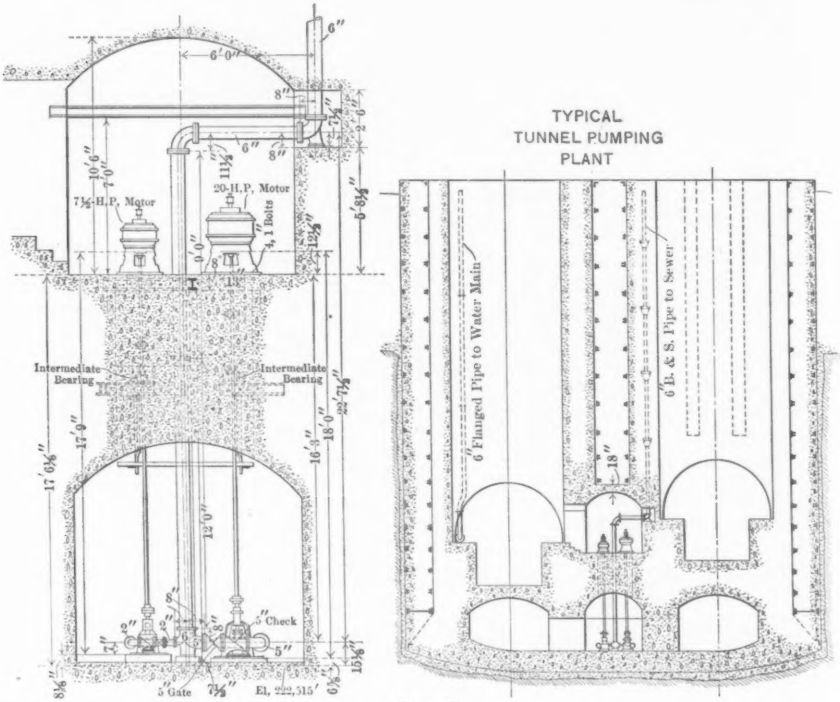

FIG. 13.

The character of the buildings would not seem to require elaborate fire protection, the Station being of unusually non-combustible construction, yet it was thought desirable to prevent any possibility of even small fires starting a panic among the passengers, in the building or in the trains under it, and, furthermore, it was desired to protect the terminal as a whole from the consequences of fire in buildings on private property surrounding it. Therefore, a very complete fire protection system was installed, both in the building and yard.

The necessary water supply is from two pumps in the 31st Street Service Plant, the fire piping system consisting of two closed loops,

one supplying the Station Building and the other the yard at the track level. These loops are of 10-in. wrought-steel pipe, the risers being of galvanized steel. The mains are carried around the building in the pipe gallery or basement covering the entire area under the public rooms, and risers are placed in special vertical chambers or flues, where they are readily accessible. The yard system leaves the Service Plant by the subways under the tracks, from which the pipes are tapped at various points under the platforms, or along the marginal retaining walls. About 3 miles of piping, varying in size from 6 to 2½ in. in diameter, were required.

There are sixteen 4-in. stand-pipes in the main building, with 83 hose connections. At the track level there are 23 hose connections on the station platforms and 12 fire-hydrants in the yard west of them. The hose equipment for the yard is stored in convenient form for quick handling in the various yard buildings, and the platform hose, in 100-ft. lengths, is coiled in special housings at the connections on the columns supporting the buildings. The building hose connections are provided also with 100-ft. lengths of 2½-in. linen hose suspended from racks, and on the roof 100 ft. of hose is stored in two centrally located hose houses. Six Siamese connections are provided at the building corners to permit the city fire engines to pump into the stand-pipe system. In addition, there is a 60-gal. chemical engine with a 500-ft. reel carriage in the baggage passageway under Seventh Avenue. The total length of hose equipment is about 10 000 ft.

The Service Plant is provided with a 4-in. loop supplying eleven hose connections and five monitor connections on the roof, for protection against fire in adjacent buildings.

Hand chemical extinguishers have been provided at various places in the Station, in all buildings in the yard, and on certain of the platforms. There are 131 of these extinguishers.

A complete local system of fire-alarm boxes covers the Station, Service Plant, Station Yard, and Sunnyside Yard. In the Station Yard there are 20 boxes located in central positions in the buildings and yard. There are also two city fire-alarm boxes in the Station Building, and these have been connected with the general city fire-alarm system. This system operates in a closed circuit, wired in loops of ten stations each, recording on three gongs, one under the main concourse, the second in the Yard Master's office, and the third

in the engine-room of the Service Plant. In addition, there are a number of punch registers and tap bells in the offices of the various station officials. The boxes are of the non-interfering, successive type of the Pennsylvania Railroad standard design. An alarm will be responded to by a special fire-brigade organization in which the men are divided into hose-wagon, chemical-engine, and stand-pipe gangs; and, in addition, men are specially designated to handle fire extinguishers.

Fire Protection in Sunnyside Yard.—In the sub-station in Sunnyside Yard there are three 500-gal. pumps; one of these is used for supplying water for cleaning cars and other yard purposes; one supplies the fire mains, maintaining a pressure of about 80 lb., and the third is held in reserve. There are fire lines throughout the yard, with the necessary hydrant connections, and these fire lines are entirely distinct from other water lines. There are fourteen fire-alarm boxes throughout the yard, and they operate visual indicators in the hose-houses and in the various offices in the buildings, and also the whistle-blowing machine. Two hose carriages are provided at central points for the fire brigade. Connection with the city fire-alarm system is made through a box installed for the purpose in the auxiliary sub-station.

Yard Lighting.

Station.—The open portion of the Station yard is lighted by 18 flaming-arc lamps, supported about 40 ft. above the tracks by brackets from the retaining walls, bridges, and certain other structures, so as to give a reasonably even distribution of light throughout. These lamps operate on alternating current at 220 volts pressure, distributed from the Station lighting mains.

Sunnyside Yard.—The general illumination of the yard is effected by forty-one 3 000-c.p. flaming-arc lamps of the long-burning type, the same as in the Station Yard. These lamps are mounted on steel poles from 200 to 350 ft. apart, having special reference to important local points. Direct current at 110 volts is supplied to the lamps from the battery-charging motor-generator sets in the auxiliary sub-station of the yard.

Manhattan Transfer.—This yard is similarly lighted by 13 flaming-arc lamps, mounted on concrete poles. These lamps operate by 220-volt 60-cycle, alternating current, furnished through transformers from the 2 200-volt signal mains, and located in Signal Cabins "N" and "S".

FIG. 1.—INTERIOR OF AUXILIARY POWER SUB-STATION, SUNNYSIDE YARD.

FIG. 2.—BATTERY-CHARGING SWITCH-BOARD, SUNNYSIDE YARD AUXILIARY POWER SUB-STATION.

Snow-Melting System.

A supply of hydro-carbon oil is carried in tanks in each of the three yards, and applied, as needed, to the switches, frogs, etc. It is set on fire at the points in question, and this furnishes sufficient local heat to melt quickly any accumulation of snow and ice. In Sunnyside Yard two 4 000-gal. storage tanks are placed underground in a remote portion of the yard, and from them the oil is piped to central points near the main switch systems at the ends of the yard. At these points the oil is drawn into small cans and thus conveyed to the switches for local application. In the Station yard, there is one 750-gal. tank west of Ninth Avenue; it receives its supply of oil through a pipe to the tank, by wagon delivery in 31st Street. From the tank, local distribution in the yard is effected in small safety cans. At Manhattan Transfer, one 1 000-gal. tank is located underground at the west end of the yard. Delivery is by a short run of piping to a central point, and thence to the switches by hand safety cans.

Electric Power System.

Electric power is needed for moving trains over the Division, and for a variety of other purposes. The requirements will hereinafter be referred to as for traction and for auxiliary purposes. Both kinds can obviously be generated most economically at a central point, and distributed over the division, by suitable transmission and distributing cable systems, to local points or sub-stations for transformation and control.

The provisions for traction power are of primary importance in the design and location of a central plant, and yet the provisions for auxiliary power must be comprehensive and complete, in order to insure uninterrupted operation of the Division as a whole. This later power serves the important operating purposes of motive power for signals, lighting the Station, tunnels, and yards, the operation of motors for various purposes about the Station, and for the ventilation and drainage of the tunnels and yard; also, for secondary purposes, in charging car batteries, as well as batteries for the telephone and telegraph system, and baggage trucks.

Selection of Traction System.—As the use of a tunnel entrance into New York City was predicated upon the use of electric traction, a statement of the conditions to be met and of the investigations leading up to the determination of a proper system, and a description

of its general characteristics, will be of interest. In 1902, when the terminal work was commenced, there were few practical examples of heavy electric traction in existence, and none of the magnitude and complexity of the proposed terminal operation. As it was realized that much experimental work would be necessary to perfect the details, a complete programme to this end was laid out some years in advance of the time when the actual installation of apparatus would be required.

Electrification of a portion of the Long Island Railroad was begun as a separate matter at about the same time, the system adopted being the so-called "third-rail," or "direct-current." This system is more properly termed the "A.C.-D.C.," that is, high-tension alternating current is generated at the power-house and transmitted as such to sub-stations, where it is lowered in potential and converted into direct current to be fed into the third-rail system. This system was practically the only one in a sufficiently advanced state of development to warrant its adoption for heavy work at the date in question. The "single-phase" alternating-current system was offered later as a possible advance in the art, and was adopted on important work, notably the New York, New Haven and Hartford Railroad from its New York connection to Stamford, Conn. This development, therefore, demanded an investigation of the relative merits of the two systems for the New York Terminal, and to this end the President appointed, as a Special Committee, the heads of the Motive Power Departments of the Pennsylvania Lines East and West, and of the Long Island Railroad, to co-operate with and report to the writer, in determining the system to be used. A systematic plan was formulated for investigating the practical behavior of the two systems, and, in order to try out certain features of the single-phase system in its application to the tunnel conditions, an experimental line 5 miles in length was constructed on a branch of the Long Island Railroad. On this line an equivalent of a tunnel section was installed and equipped with a high-tension overhead conductor; various contact devices designed for sparkless collection of current were tried; and a multiple-unit motor-car equipment and a special type of A.C. locomotive were tested. The test car and locomotive, as well as much other apparatus, were kindly supplied by the Westinghouse Electric Company.

The results of the tests cannot be given here in detail, but they were reassuring in some respects and disappointing in others. In the

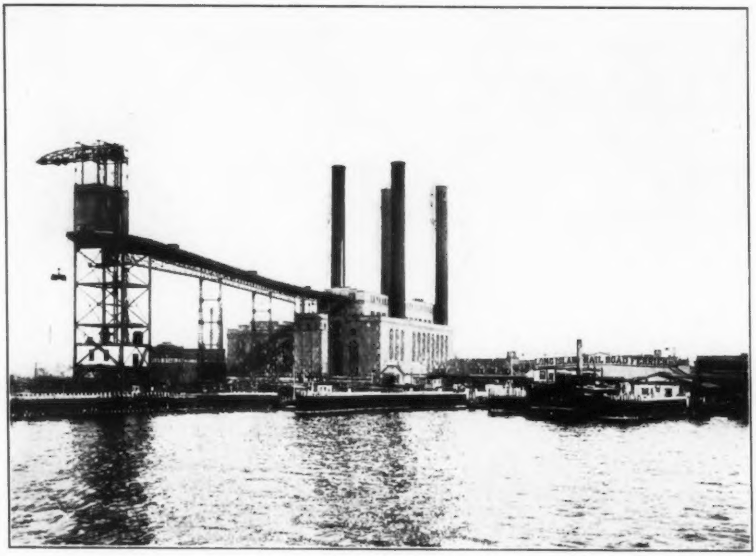

FIG. 1.—LONG ISLAND CITY POWER-HOUSE.

FIG. 2.—ENGINE-ROOM, LONG ISLAND CITY POWER-HOUSE. 3 000-KW. AUXILIARY POWER GENERATORS IN FOREGROUND.

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main, they demonstrated the need of considerable further experimental work to adapt the system to the tunnel and yard conditions. The recommendation of the committee was for the adoption of the "Direct Current" (A.C.-D.C.) system, interchangeable with that used on the Long Island Railroad, for the following main and important reasons:

First, reliability, from the start; second, freedom from complication in interchange with the Long Island traction system and the Newark Rapid Transit line through the Hudson and Manhattan tubes into Church Street, New York; and third, less expenditure involved at present and for some time to come. The above reasons, it should be remembered, apply only to this special installation. No broad generalization is intended for other traction projects, where local conditions must govern.

Load Conditions.—The load conditions include both traction and auxiliary power; and, in case of traction power, the requirements for the Long Island Railroad. Considering all these requirements, the load center was at a point adjoining the railway lines in Long Island City and not far from the water-front of the East River. It was at first proposed to establish two power-houses, each to relay the other, under which plan the location of one house would fall on the New York side of the Hudson River and the other at a point not far from Jamaica. Neither of these locations was altogether suitable, on account of difficulty in procuring the site in the first mentioned place, and the absence of water for condensing purposes in the other. Furthermore, it was found that, in dividing the load between two plants, the call on neither one would be sufficient to give reasonable economy in operation; the relaying feature, therefore, of the two power-houses, in the case of the disablement of one of them, was the only important point in favor of such a plan. Experience has shown that a properly designed plant, provided with suitable safeguards against a general breakdown from an accident to a part of the machinery, and with duplicate cable connections to the outside, can be depended on to furnish an uninterrupted supply of power, except for very infrequent and short interruptions, in which cases power supply can be restored in less time than it would take to obtain full relief from a second plant, especially where both are only of moderate size. Furthermore, in a great electric traction center, such as the City of New York, it is possible to establish emergency connections with other power-plants. In view of these

considerations, therefore, it was determined to establish only one power-house for present operation, and to equip it to supply power for both traction and auxiliary purposes.

For an emergency supply of power for traction, it has been arranged to connect the general traction system at three different points with the power-plants of three companies, namely, the Public Service Corporation, in New Jersey, the Hudson and Manhattan Railroad Company, in New York, and the New York and Queens County Railway, in Long Island City.

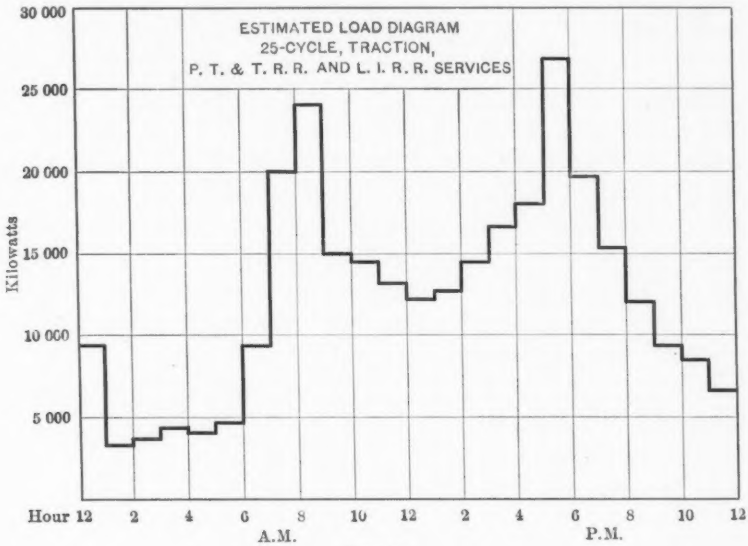

FIG. 14.

As an emergency supply for auxiliary power purposes the Service Plant, described elsewhere, is equipped with generating apparatus capable of supplying all the auxiliary power required for the entire system. The combined load conditions for all services are shown in the curves, Figs. 14 and 15; and the central power-plant has been designed to care for these loads with sufficient reserve for ordinary emergencies and a limited amount of spare capacity in the building for growth of traffic; but it has not been designed to take care of a general extension of the electric traction system to other Pennsylvania lines. When such extensions are made, additions can be made to the present power-house building, or, preferably, a new power-house can be erected on the New Jersey side.

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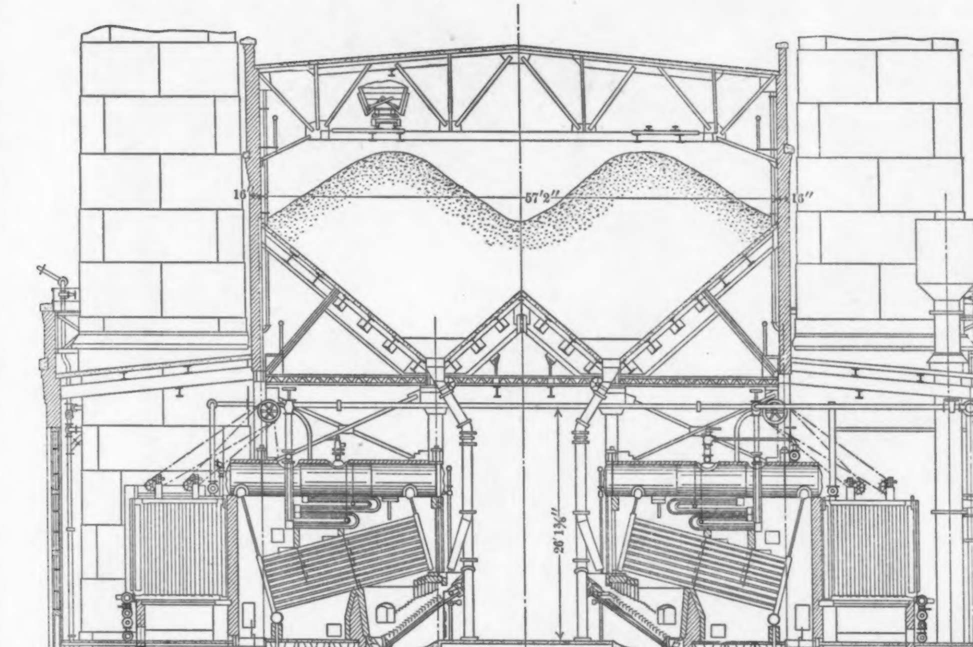
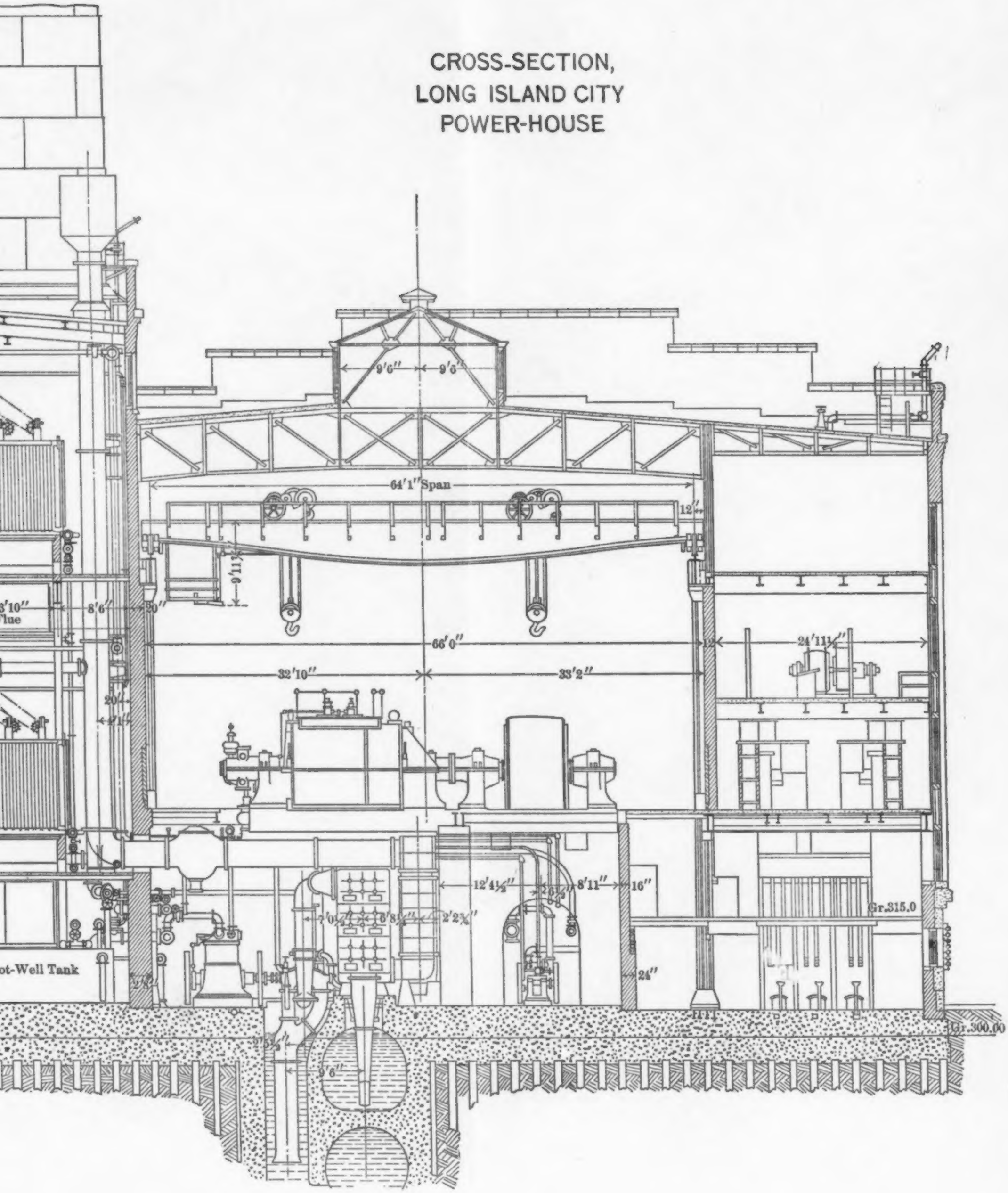

16"
57'2"
16"
26'1⅜"
8'6"
13'10"
Flue
10'3"
9'9"
9'9"
102'8"
19'3"
13'10"
Flue
8'6"
3'6"
3'6"
26'3⅜"
Flue
Flue
Hot-Well Tank
7'0"
5'4½"
5'4½"

PLATE XCIII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.

CROSS-SECTION,
LONG ISLAND CITY
POWER-HOUSE



Long Island City Power-House.

—The Long Island City Power-house, Fig. 1, Plate XCII, ·t River, in Long Island City, a location nearly central ι load conditions, close to the tunnel lines, and requiring ble connections to the tunnels, and thence over the Ter- . n.

ing of this plant was undertaken some years prior to the of the tunnel system, in order to care for the electrified lii ıpletio Long Island Railroad into the City of Brooklyn, follow-

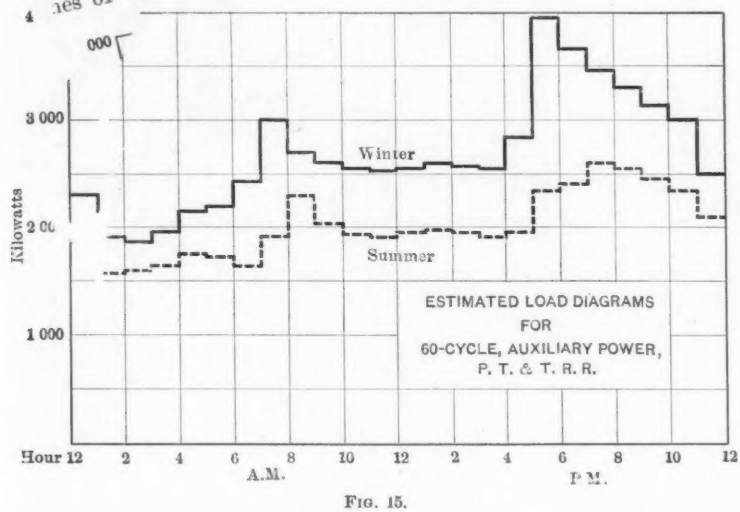


Fig. 15.

ing the completion of the Atlantic Avenue Improvement of that road; and power from it for this operation has been furnished since 1905. Recently additional equipment has been added for the tunnel operation.

Foundations.—The site was partly under water, with solid rock at a depth of from 35 to 60 ft. below high water, the intermediate strata being of clay, sand, and gravel. It was decided to use a pile foundation, overlaid with a monolithic concrete base. The piles were spaced at 2-ft. centers, on an average, and carry a load of 12 tons each; 9 100 piles in all were used. They were cut off below the extreme low-water line and overlaid with a concrete mattress 6 ft. 6 in. thick, in which about 18 000 cu. yd. of concrete were required.

Building.—The building is a steel-frame structure with brick curtain-walls. It is of fire-proof construction throughout, with windows of wire glass in metal frames. The boiler-room is double-decked; the engine-room adjoins, with the switching galleries at the side (see Plates XCIII and XCIV). The coal is stored above the boiler-room in pockets having 5 200 tons capacity. The bunkers are of reinforced concrete, with partitions dividing the storage space. There are four stacks, carried from the base independently of the house. They are 233 ft. high above the grates of the upper tier of boilers, or 275 ft. from the base, with an interior diameter of 17 ft. They are of steel, brick-lined to the top. The coal-handling plant is designed to deliver coal into the power-house pockets from barges at the water-front dock, although delivery from cars may be made if required.

Fire Protection.—A complete fire protection system has been provided, chiefly because of the possible hazard from adjoining buildings. The stand-pipe system consists of 6-in. risers at the corners of the building connecting to seven hydrants on the roof with monitor nozzles. There are also two nozzles on the bridge of the coal tower and two at the extreme end to protect the dock property. Six Siamese steamer connections are provided at the street level, and may be used for hose connections or for supplying the stand-pipe system from the city fire department steamers. In the basement there are two duplex fire pumps, each having a capacity of 1 000 gal. per min., each with a 10-in. suction from the river, and delivering into the header for the building supply. The coal bunkers are provided with pipe connections for flooding in case of necessity.

Coal Handling.—A hoisting tower on the deck is connected with the house by a bridge, 500 ft. in length; with its deck 107 ft. above the dock; the tower is 170 ft. high. Coal is hoisted in 2-ton self-loading buckets, at a speed of 1 000 ft. per min., to the weighing and crushing machinery in the tower, from which it is delivered into the cars of a cable railway on the bridge. The machinery is capable of unloading and delivering into the bins 150 tons per hour, and, for the past three years, the average cost of handling, for labor and maintenance, has been 5.40 cents per ton. Ashes are delivered into a bin outside of the building and over a railroad track, by a telpherage system which hoists and transports the ash cars from the boiler-room basement.

PLAN OF ENGINE-ROOM, LONG ISLA

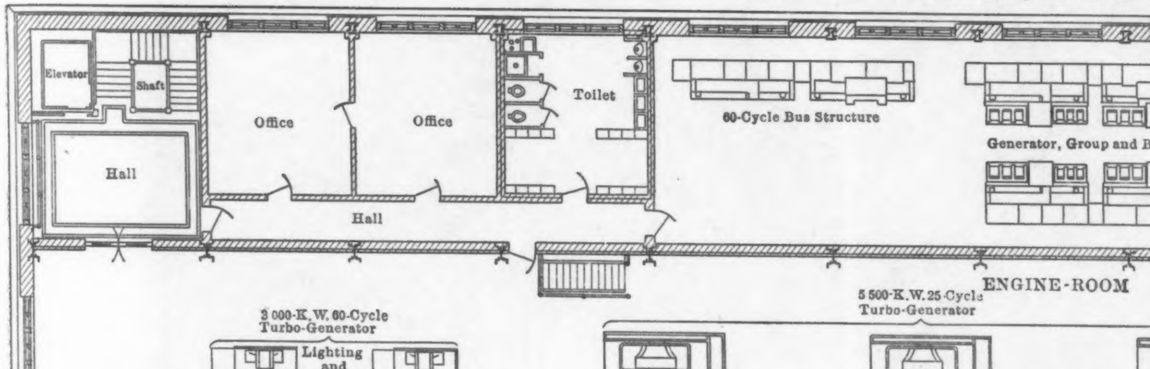
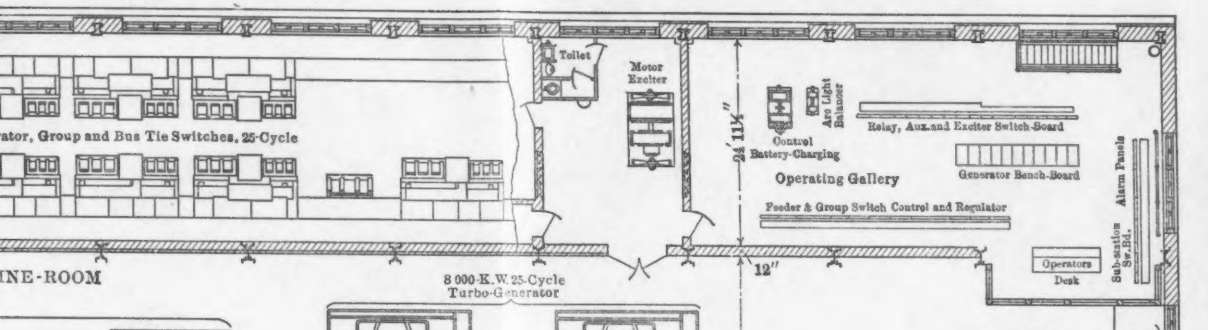


PLATE XCIV.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.

LONG ISLAND CITY POWER-HOUSE.



Boilers.—The present equipment consists of thirty-two boilers, each of 564 h.p., sixteen on each floor, of Babcock and Wilcox water-tube type, built for a working pressure of 200 lb. per sq. in., and fitted with superheaters for 150 lb. of superheat, and mechanical stokers. At the rear of the boilers are placed the economizers, one for each two batteries of boilers.

Turbo-Generators.—Westinghouse-Parsons steam turbines drive directly the three-phase, A.C., 11 000-volt, 25-cycle, revolving-field generators. The equipment initially installed to operate the Long Island Railroad traction system consisted of three units of 5 500 kw. each, at 750 rev. per min. To care for the increase of load from the Terminal operation, two units were recently added. These are double-flow turbines, direct-coupled to 11 000-volt, three-phase generators of 8 000 kw. capacity at 750 rev. per min.

Two 3 000-kw. turbo-generators of the same type (Fig. 2, Plate XCII), generating three-phase current at 11 000 volts and 60 cycles, have been provided for auxiliary power purposes; these units relay similar units in the 31st Street Service Plant, and are used either for emergency, or in summer when exhaust steam from the latter plant is not needed for heating the Station Building. There is space in the present building for an additional unit of 8 000 kw., with the necessary boilers and accessories.

Condensers.—All the turbines are condensing. The condensers for the three original 5 500-kw. units are of the "surface" type, and were selected in order to reclaim the boiler feed-water, as water had to be purchased and was quite expensive. Injection water is obtained from the East River. As was expected, the maintenance cost of these surface condensers proved to be rather high, and when the additional units were installed an opportunity to use "jet" condensers economically was afforded by the discovery of an ample supply of good water in Sunnyside Yard. This supply is estimated to be sufficient to care for all power-house boiler feed-water as well as the yard requirements. The four new units were provided with jet condensers of the Le Blanc type.

Exciters.—Three separate sources of current are provided for exciting the fields of the generators: these consist of, two steam-driven units, one motor-driven unit, and a storage battery.

Switching Apparatus.—The switching apparatus is in a separately-

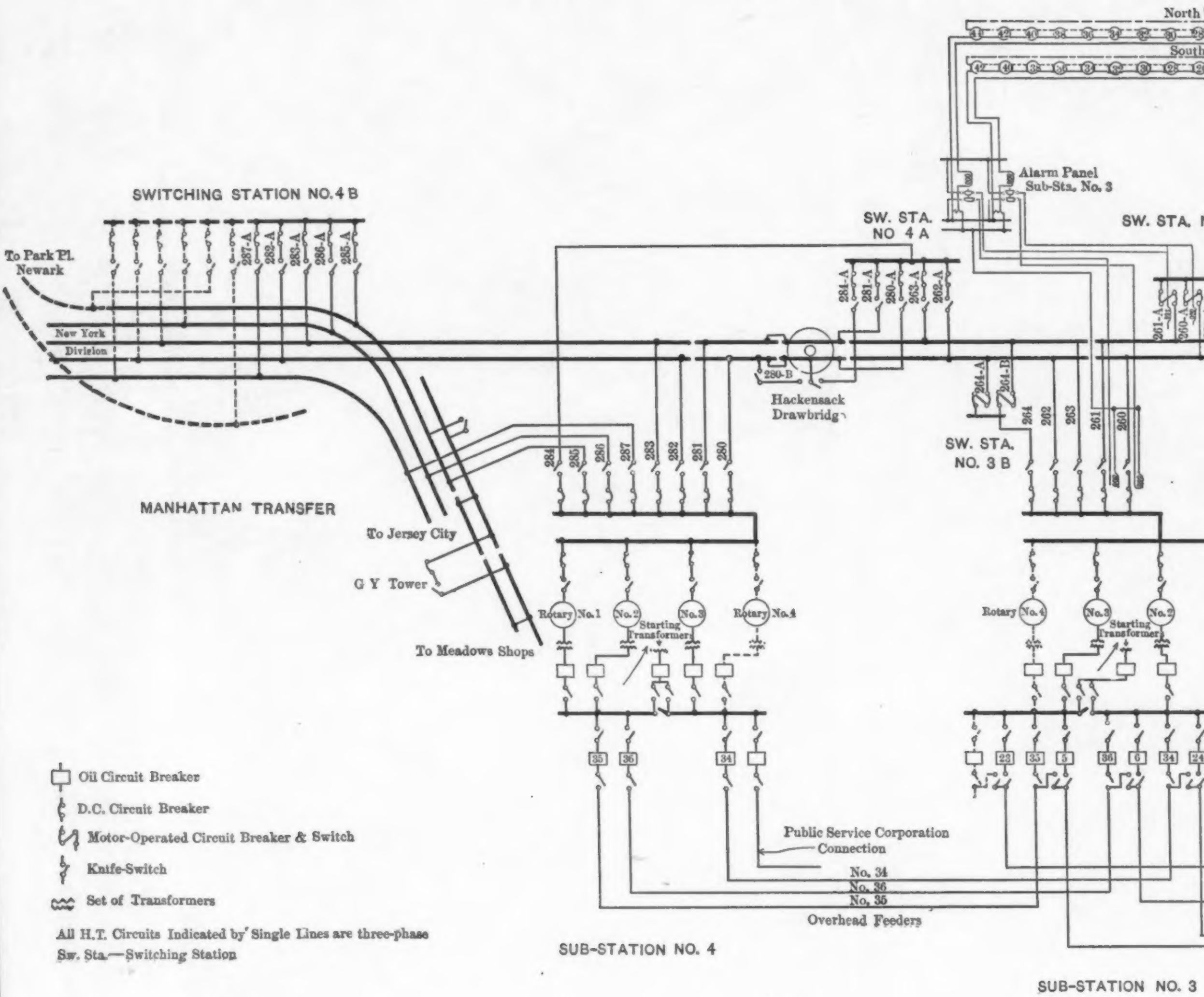
housed gallery of three floors and a basement. The various cables are brought out of the conduits at the basement floor in cubicles to the switches on the floor above. Transformers for lighting purposes and for the motor-driven exciter are also located on the basement floor, as well as the storage battery for emergency excitation and for operating the electrically-controlled switch-board.

On the first floor are mounted the main generator-switches and the individual feeder-switches. On the second floor are the selector-switches to the generators, and the selector-group-switches for the feeders; also the main bus-bars and tie-switches for the same. On the third floor are installed the main controlling switch-board panels, instruments, etc., for controlling all generator-feeder- and group-switches. This floor also contains the motor-driven exciter and battery-charging booster, and at one end there is a separate control board for controlling the sub-station machinery. Near the main switches a glass-enclosed bay is provided, from which is afforded a full view of the engine-room containing the turbo-generators and rotary converters.

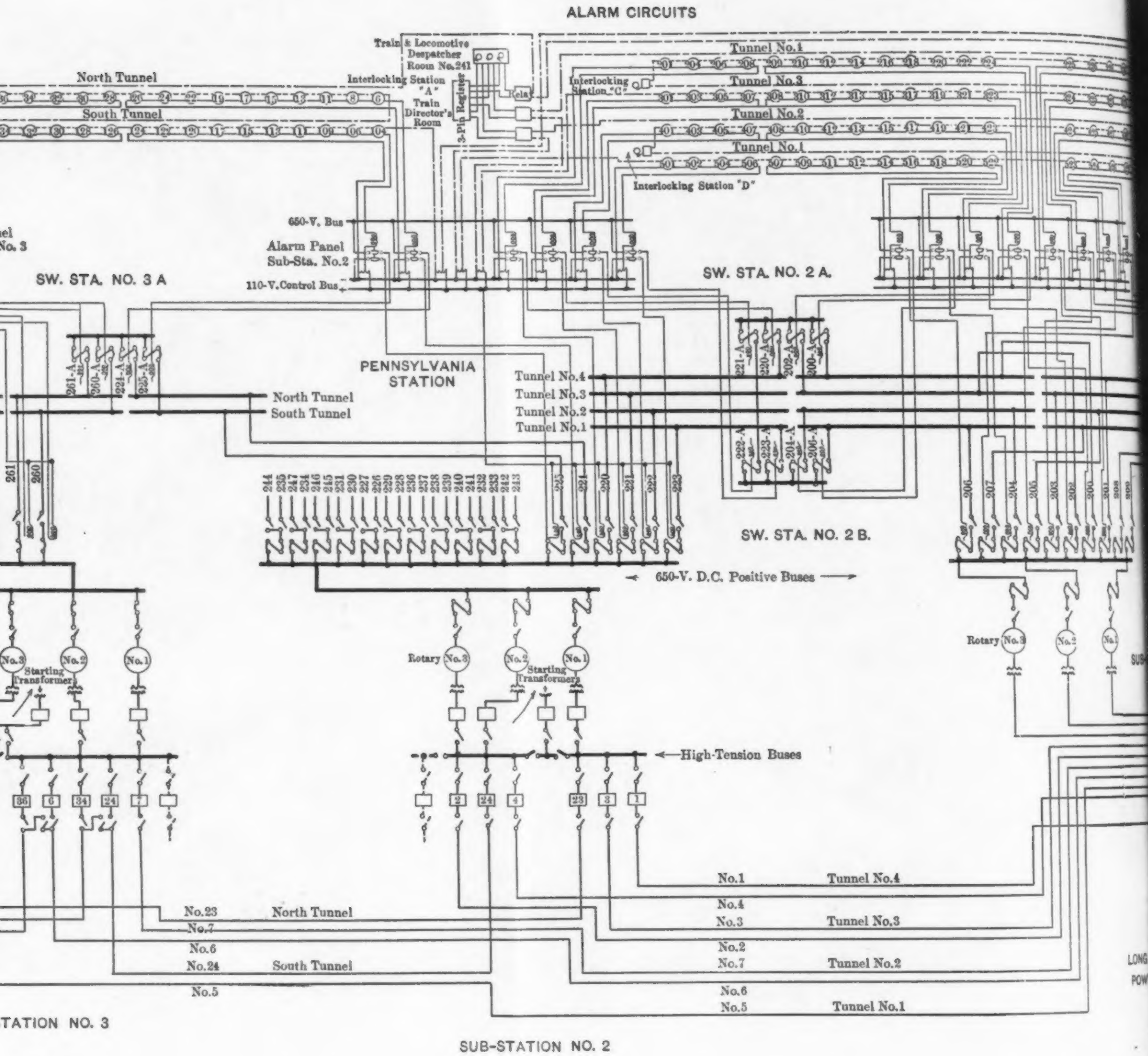
The switching arrangement consists in providing an open-ring bus structure, divided into three sections, controllable by the operator through oil-switches. The three 5 500-kw. generators and all the switch feeder groups are arranged to connect to either of the two end loops, and two 8 000-kw. generators are connected to the middle sections of the ring.

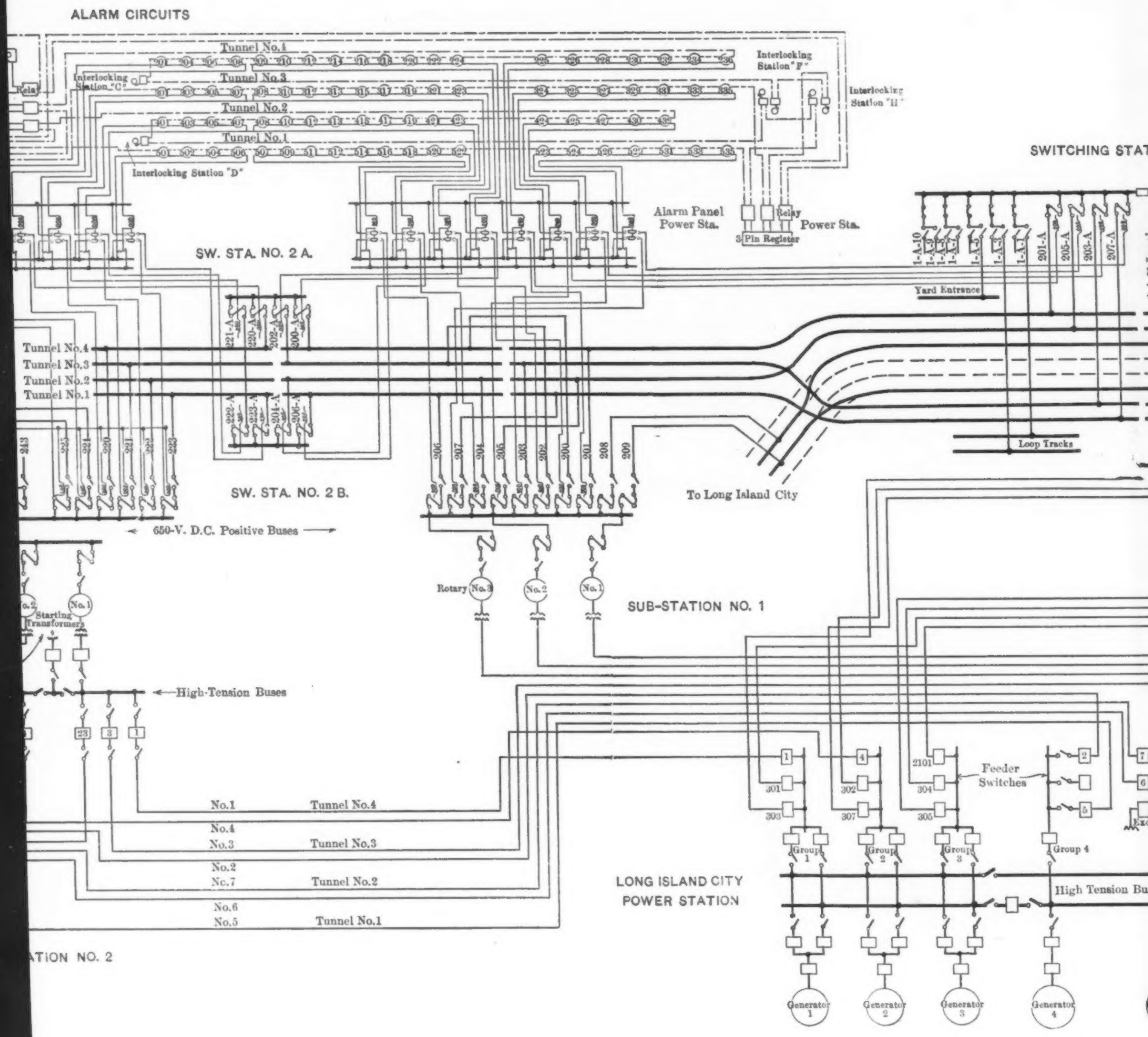
All feeders are arranged in groups, usually three feeders per group, and connected through the oil circuit-breakers to group bus-bars, which in turn are connected to the main bus-bars through selector-group-switches. All the remote-controlled switches and circuit-breakers are of the oil type, having the contact immersed in the oil chamber at all times, which effectively extinguishes an arc when the switch is broken under load.

The feeder cables which lead to the Long Island Railroad, where the transmission is in the main overhead, and where short-circuits are consequently more apt to occur, are provided with auxiliary switches having resistance contact, which, during the opening of the switch, serves first to cut resistance in series, thus reducing the current before the actual break occurs. The neutral points of the generator winding are connected to a neutral bus-bar, which is grounded through a suit-



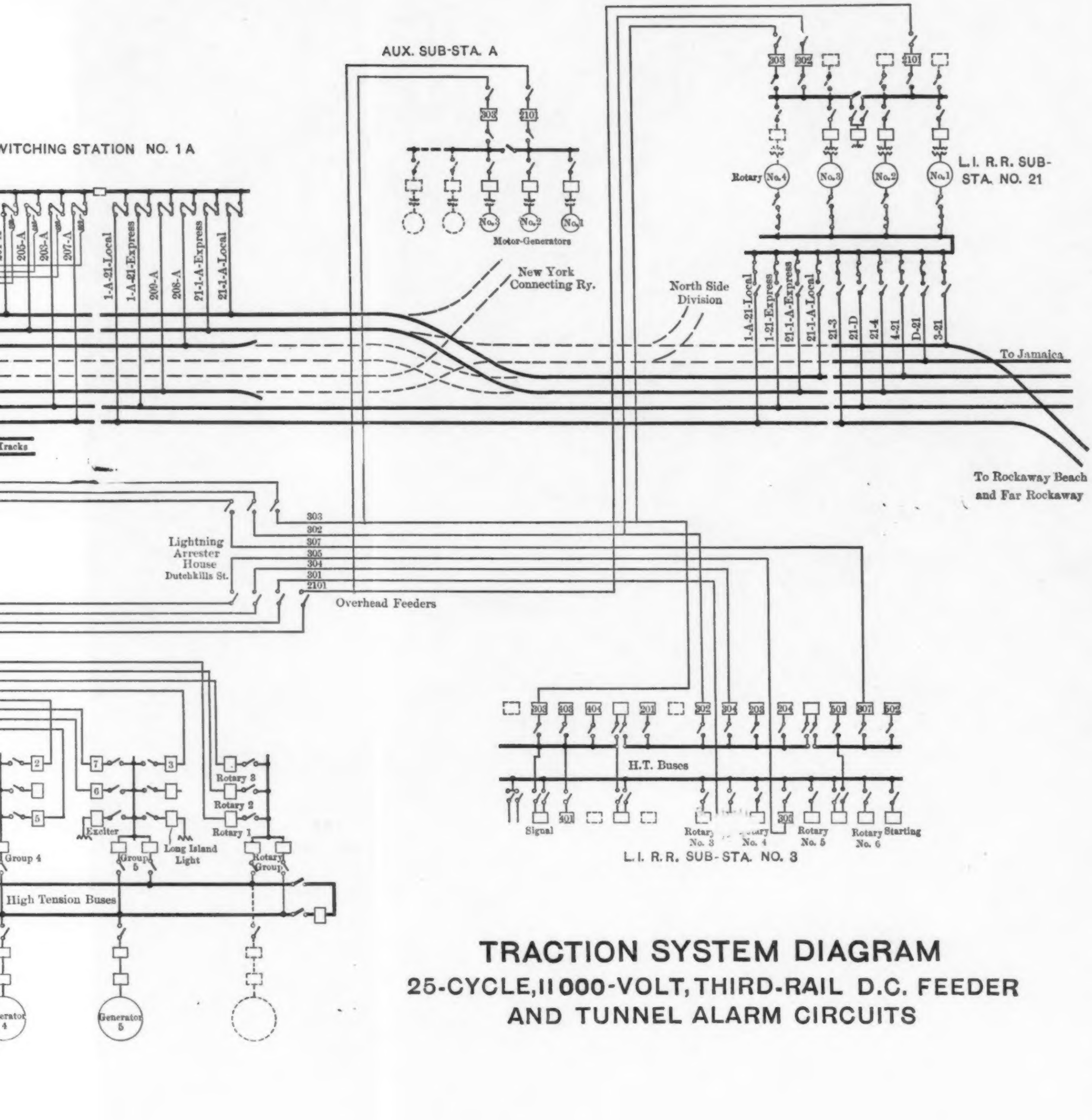
SWITCHING STATION NO. 4 B
To Park Pl. Newark
New York Division
287-A
282-A
283-A
286-A
285-A
MANHATTAN TRANSFER
To Jersey City
G Y Tower
To Meadows Shops
SW. STA. NO 4 A
284-A
281-A
280-A
263-A
262-A
280-B
Hackensack Drawbridge
Alarm Panel Sub-Sta. No. 3
North
South
SW. STA.
SW. STA. NO. 3 B
264-A
264-B
264
262
263
261
260
261-A
260-A
284
285
286
287
283
282
281
280
Rotary No. 1
No. 2
No. 3
Rotary No. 4
Starting Transformers
Rotary No. 4
No. 3
No. 2
35
36
34
23
5
6
24
Public Service Corporation Connection
No. 34
No. 36
No. 35
Overhead Feeders
SUB-STATION NO. 4
SUB-STATION NO. 3
Oil Circuit Breaker
D.C. Circuit Breaker
Motor-Operated Circuit Breaker & Switch
Knife-Switch
Set of Transformers
All H.T. Circuits Indicated by Single Lines are three-phase
Sw. Sta.—Switching Station


ALARM CIRCUITS
Train & Locomotive Despatcher Room No.241
Interlocking Station "A"
Train Director's Room
3-Pin Register
Relay
Interlocking Station "C"
Interlocking Station "D"
North Tunnel
South Tunnel
Tunnel No.4
Tunnel No.3
Tunnel No.2
Tunnel No.1
650-V. Bus
Alarm Panel Sub-Sta. No.2
110-V. Control Bus
SW. STA. NO. 3 A
SW. STA. NO. 2 A.
PENNSYLVANIA STATION
SW. STA. NO. 2 B.
650-V. D.C. Positive Buses
Rotary
Starting Transformers
High-Tension Buses
No.23
No.7
No.6
No.24
No.5
No.1
No.4
No.3
No.2
TATION NO. 3
SUB-STATION NO. 2



ALARM CIRCUITS
Tunnel No.4
Tunnel No.3
Tunnel No.2
Tunnel No.1
Interlocking Station "C"
Interlocking Station "D"
Interlocking Station "F"
Interlocking Station "H"
Relay
Alarm Panel Power Sta.
Power Sta.
3 Pin Register
SW. STA. NO. 2 A.
SW. STA. NO. 2 B.
SWITCHING STAT
Yard Entrance
Loop Tracks
To Long Island City
650-V. D.C. Positive Buses
Rotary No.3
No.2
No.1
Starting Transformers
SUB-STATION NO. 1
High-Tension Buses
Feeder Switches
Group 1
Group 2
Group 3
Group 4
High Tension Bu
LONG ISLAND CITY POWER STATION
Generator 1
Generator 2
Generator 3
Generator 4
ATION NO. 2

PLATE XCV.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.



TRACTION SYSTEM DIAGRAM
25-CYCLE, 11 000-VOLT, THIRD-RAIL D.C. FEEDER AND TUNNEL ALARM CIRCUITS

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able grid resistance adjusted to limit the flow of current when the line becomes grounded.

The 60-cycle switching apparatus is similar in every way to the 25-cycle equipment, except that a single set of bus-bars is used and is provided with knife-switches for dividing the two sections in emergencies.

The machinery equipment of this power-house, excluding the traction sub-station (given elsewhere), may be concisely listed as follows:

Coal pocket at top of building; capacity, 5 200 tons.
Thirty-two 564-h.p. water-tube boilers, 200 lb. working pressure, 125° superheat. Space for 16 additional 564-h.p. boilers.
Boilers equipped with 32 Type D, Roney stokers, 150 in. wide, 12 grates deep.
Two 8 000-kw., 11 000-volt, 3-phase, 25-cycle, turbo-generators, and space for one additional, for traction power.
Three 5 500-kw., 11 000-volt, 3-phase, 25-cycle, turbo-generators, for traction power.
Two 3 000-kw., 11 000-volt, 3-phase, 60-cycle, turbo-generators, for auxiliary power.
One 200-kw., motor-driven exciter.
One 50-kw., motor-driven exciter.
Two 200-kw., turbine-driven exciters.
One 600-ampere-hour, storage battery of 110 cells.
One Tirrill regulator for 25-cycle generators.
One Tirrill regulator for 60-cycle generators.
Three 175-kw., oil-cooled transformers.
Thirteen 1 200-ampere, 11 000-volt, Type C, oil circuit-breakers.
Thirty-nine 600-ampere, 11 000-volt, Type C, oil circuit-breakers.

Switch-board:

One bench-board, eight panels, for control of 25-cycle generators; and three panels for control of 60-cycle generators.
Six 25-cycle, generator panels.
Two 60-cycle, generator panels.
Two 25-cycle, station panels.
One 60-cycle, station panel.

Transmission.

The transmission of electrical energy from the power-house is entirely 11 000-volt, three-phase, alternating current, the traction power being 25-cycle and the auxiliary power 60-cycle (see Plate XCV). The distribution of all power is by three-conductor, paper-insulated, lead-

covered cables, drawn into the conduits between the switch-boards and the shafts communicating to the tunnels.

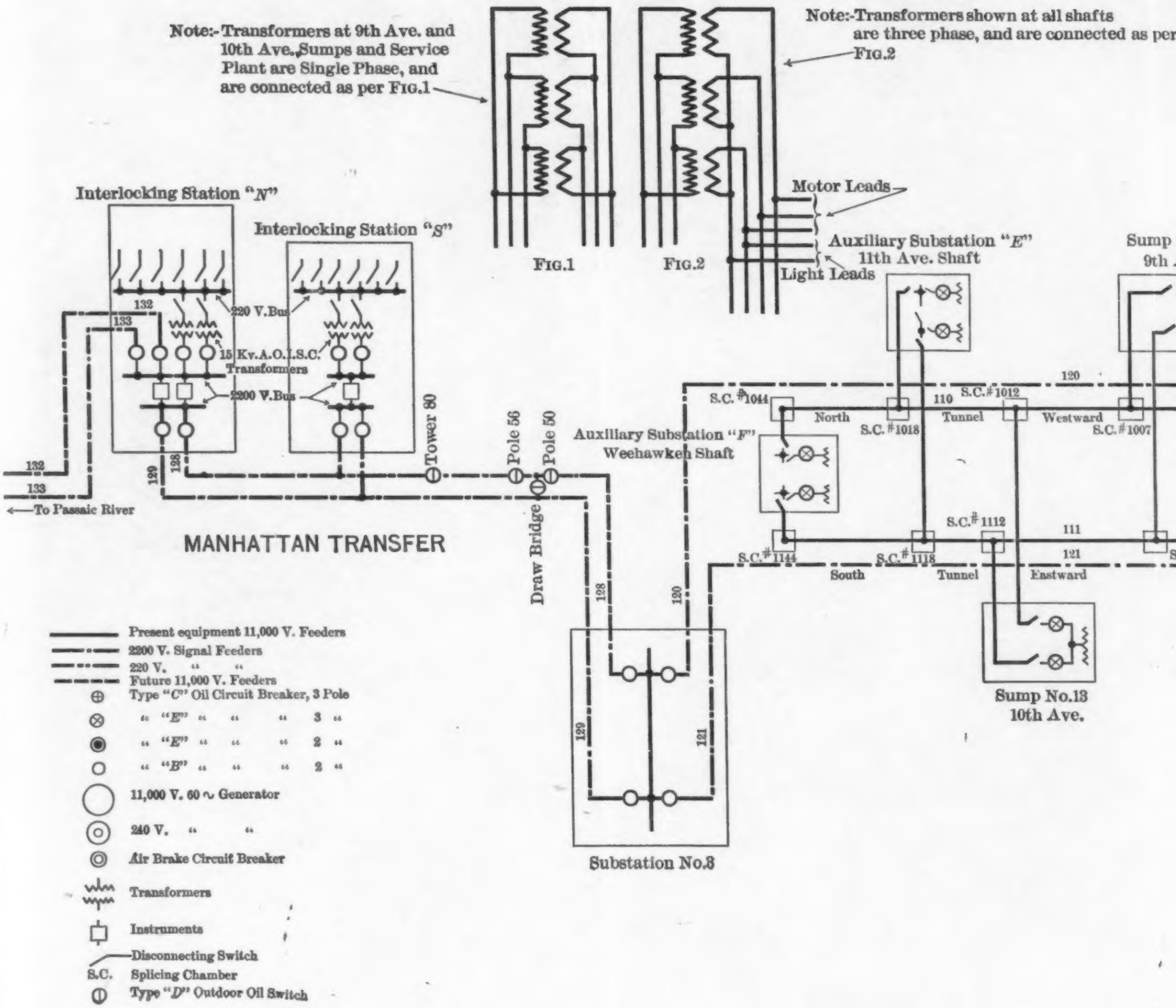
Four power cables run from the Power-House direct to Service Plant Sub-station No. 2; three additional power cables run direct from the Power-House to Sub-station No. 3, at Hackensack Portal, New Jersey; and two extension cables connect Sub-stations Nos. 2 and 3. At Sub-station No. 3 these feeders leave the house in three 3-phase open-wire circuits on the steel pole line, crossing the Meadows and terminating at Sub-station No. 4, at Harrison. All these 25-cycle feeders are of 250 000 cir. mils section per conductor.

Power for the Long Island Railroad is supplied in a similar manner through seven high-tension cables laid in the conduit system to an arrester-house at the west end of Sunnyside Yard, from which open circuits are continued, through suitable lightning arresters, overhead on a steel pole line through the yard and thence to various sub-stations on the Long Island electric traction system.

As before mentioned, the potential of the high-tension lines is from 11 000 to 12 000 volts, this being sufficiently high for economical transmission for the distance at which power is at present used on the Pennsylvania and the Long Island systems; it is a suitable potential for direct generation without using step-up transformers, and is a commercial voltage for insulated cable. To provide for the contingency of an extension of the traction system on the New Jersey side beyond Newark, or for tie lines to a future power-house, space has been provided on the pole line for additional circuits suitably spaced for 33 000-volt transmission. In the event of such requirement, it is intended to provide step-up transformers in the Hackensack Portal Sub-station, and a 33 000-volt line westward therefrom.

Auxiliary power (60 cycles) is transmitted from the power-house through the tunnels to the Service Plant in a similar way (see Plate XCVI), by four cables, two of these going to the Service Plant direct, and two to the tunnel shaft-houses for tunnel lighting and miscellaneous power. The two cables to the Service Plant have conductors of No. 00 B. & S. section, and those to the tunnel shaft houses have conductors of No. 4 B. & S. section.

As there are four tunnels under the East River, the 25-cycle and 60-cycle cables are subdivided into groups and distributed through the tunnels so that there is a duplication of routes, as well as cables, for



Note:-Transformers at 9th Ave. and 10th Ave. Sumps and Service Plant are Single Phase, and are connected as per FIG.1
Note:-Transformers shown at all shafts are three phase, and are connected as per FIG.2
FIG.1
FIG.2
Motor Leads
Light Leads
Auxiliary Substation "E" 11th Ave. Shaft
Sump
9th
Interlocking Station "N"
Interlocking Station "S"
220 V.Bus
15 Kv.A.O.I.S.C. Transformers
2200 V.Bus
132
133
128
129
Tower 80
Pole 56
Pole 50
Draw Bridge
Auxiliary Substation "F" Weehawken Shaft
S.C. #1044
S.C. #1018
S.C. #1012
S.C. #1007
S.C. #1144
S.C. #1118
S.C. #1112
110
120
111
121
North
Tunnel
Westward
South
Tunnel
Eastward
←To Passaic River
MANHATTAN TRANSFER
Substation No.3
Sump No.13 10th Ave.
Present equipment 11,000 V. Feeders
2200 V. Signal Feeders
220 V. " "
Future 11,000 V. Feeders
Type "C" Oil Circuit Breaker, 3 Pole
" "E" " " " 3 "
" "E" " " " 2 "
" "B" " " " 2 "
11,000 V. 60 ~ Generator
240 V. " "
Air Brake Circuit Breaker
Transformers
Instruments
Disconnecting Switch
S.C. Splicing Chamber
Type "D" Outdoor Oil Switch

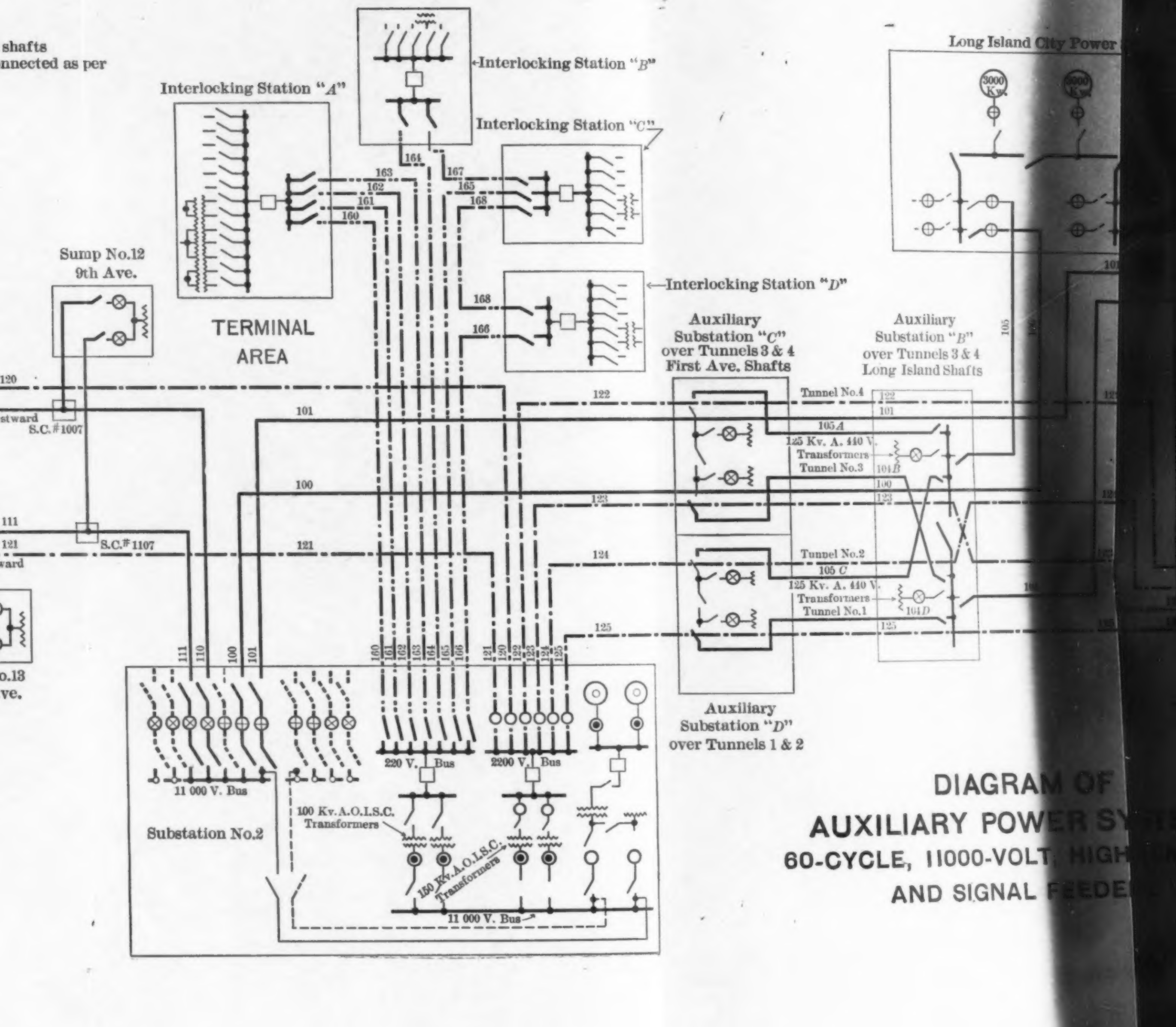
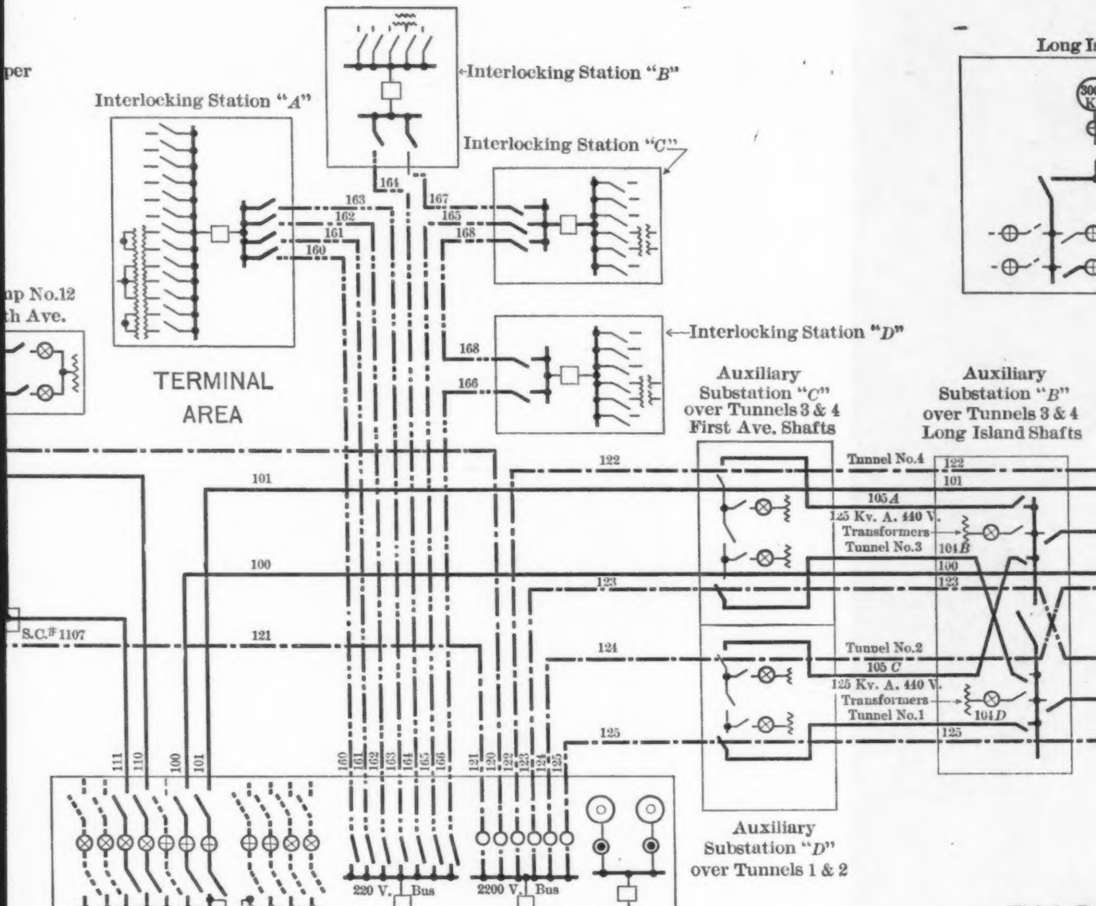


DIAGRAM OF
AUXILIARY POW
60-CYCLE, 11000-VOLT
AND SIGNAL



DIAG
AUXILIARY P
60-CYCLE, 11000-V
AND SIGN

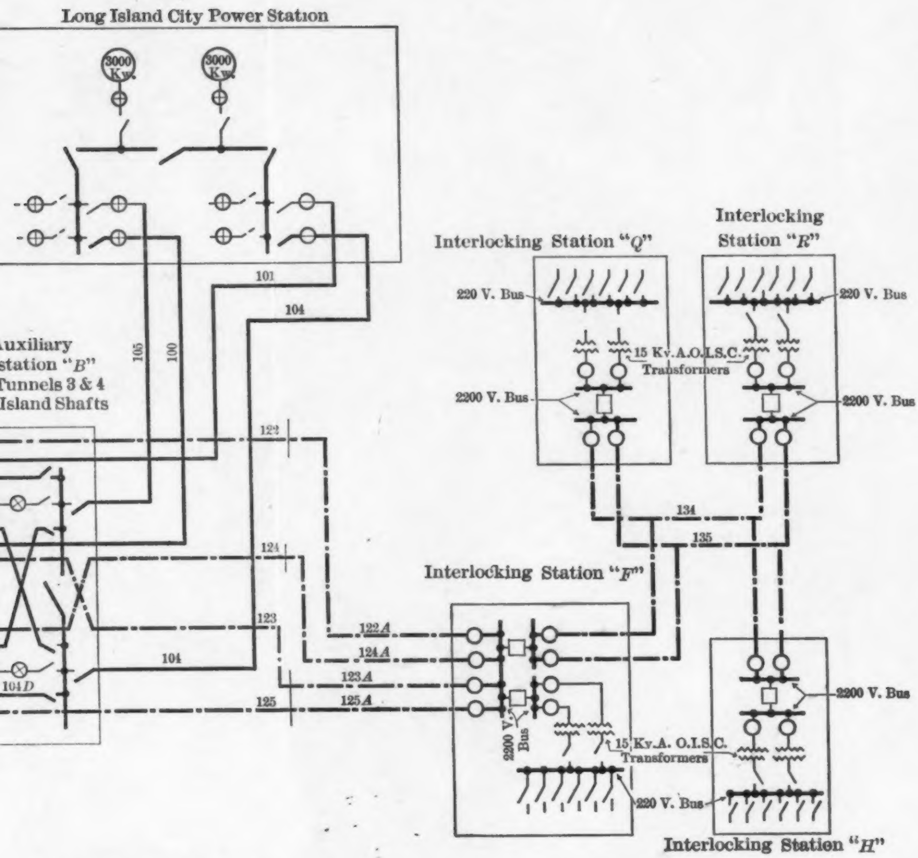


DIAGRAM OF
IARY POWER SYSTEM
, 11000-VOLT, HIGH-TENSION
ND SIGNAL FEEDERS

all services. Between the power-house and the shafts the conduit lines are divided into two groups with separate manholes, in each of which approximately half of the cables of each type are run. Similarly, the cables for the Long Island transmission are kept entirely distinct from those of the tunnel operation.

Furthermore, in providing cables for various purposes, by the above arrangement each sub-station has three or more traction power cables, two of which are ample to carry the load, and the auxiliary power cables are in duplicate; thus there is a margin of from 30 to 50% in the capacity of the cables, to provide against break-down.

Each cable is provided at each end with a high-tension, automatic, oil circuit-breaker, disconnecting switches, and section bus-bars, so as to give complete control of the cables when in operation and permit of the isolation of one or more cables or sections of the bus-bars for repair purposes without interfering with the operation of the remaining cables and sections.

At splicing chambers in the duct system, and where the cables are exposed in the sub-stations, the lead sheathings of the cables are connected and grounded at intervals for protection against electrolysis.

Traction Sub-stations.

There are four traction sub-stations for the Terminal Division, and their locations have been fixed with reference to the loading requirements and by certain physical conditions along the line.

No. 1 has been placed in the Power-House, and supplies the East River tunnel lines, Sunnyside Yard, and the Long Island tracks through the yard to a point where the load is taken by the first sub-station of the latter road. This location secures economy in first cost and operation of the direct-current supply for this section. No. 2, in the Service Plant, adjoins the main yard, and is centrally located for movements in the yard, the East River and North River Tunnels. No. 3 is at the Hackensack Portals, because of the desirability of having the power supply near the tunnel grades, and in order to permit of changing from underground cables to overhead line construction at the first point where such a change was permissible; it obviates separate provisions for housing switching apparatus and for lightning protection at a point between sub-stations. No. 4 is at the junction of the Terminal and the New York Divisions, making it available for

supplying power, not only to the Terminal Division, but for the Rapid Transit line between Newark and Jersey City, in connection with the Hudson and Manhattan Company's tunnels to Church Street, Manhattan. Plate XCVII shows the interior of this sub-station.

TABLE 9.—SUB-STATIONS.

Sub-Station.	No. 1.	No. 2.	No. 3.	No. 4.
Location	Long Island City Power-house.	Service Plant.	Hackensack Portal.	Harrison.
Incoming high-tension feeders	3	4	5	3
Outgoing high-tension feeders	....	2	3	....
High-tension lightning arresters	....	....	3	3
Size of rotaries	2 000-kw., 6-Ph.	2 000-kw., 6-Ph.	2 000-kw., 6-Ph.	2 000-kw., 6-Ph.
Number of rotaries	3	3	3	3
D. C. voltage	650	650	650	650
Main transformers	750-kw., air-blast.	750-kw., air-blast.	750-kw., air-blast.	750-kw., air-blast.
No. of transformers	9	9	9	9
Starting transformers	125-kw., OISC.	75-kw., OISC.	75-kw., OISC.	75-kw., OISC.
No. start. transformers	3	3	3	3
No. H. T. panels	0	4	8	3
No. rotary panels	4	4	4	4
No. aux. power panels	....	....	2	2
No. DC feeder panels	11	19	6	9
No. load panels	1	1	1	1
No. blowers	2-20 000 cu. ft.	2-20 000 cu. ft.	2-20 000 cu. ft.	2-20 000 cu. ft.
No. motor-generating sets	....	....	1	1
Size control battery	....	....	20-amp., 55-cell	20-amp., 55-cell.

Equipment.—In each of the sub-stations there are installed high-tension switching apparatus, step-down transformers of the air-blast type, and three 2 000-kw. rotary converters for converting the alternating into direct current for traction purposes, also, the necessary low-tension switching apparatus for controlling the out-going direct current to the various third-rail feeders. The normal direct-current voltage is 650, and the machines are designed with special reference to the fluctuating nature of the traction load, and will operate up to 200% overload. Two of the rotaries in each sub-station will carry the load on that sub-station, the third being in reserve, and space is provided in the buildings for an additional machine when required.

The apparatus has been arranged with special reference to economy in the number of attendants required in the operation, the machinery and switch-boards being on the same floor. In all sub-stations the high-tension circuit-breakers are remote-controlled electrically from the switch-boards, and in Nos. 1 and 2 the direct-current switches and breakers are operated by small motors and are remote-controlled.

PLATE XCVII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
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TRACTION SUB-STATION AT HARRISON.

Auxiliary Power Sub-stations.

There are six sub-stations at convenient points for supplying the tunnels and yards with auxiliary power for the various purposes described elsewhere; they have been designated by letters in order to distinguish them from the traction sub-stations.

Auxiliary Sub-station "A."—Sub-station "A" is in Sunnyside Yard, and is the only one using 25-cycle current. The 11 000-volt, 3-phase, 25-cycle power is obtained from two of the overhead Long Island Railroad feeders passing the yard; it is stepped down in suitable transformers, and converted by three 250-kw. motor-generators into 110 to 220-volt, 3-wire, direct-current power for charging train batteries, yard and building lighting, operation of elevators, and for miscellaneous motors for driving machine tools, cranes, refrigerating plant, etc.

TABLE 10.—AUXILIARY POWER SUB-STATIONS.

Designation.	"A"	"B."	"C."	"D."	"E."	"F."
Location	Sunnyside Yard.	L. I. Shaft House, 3 and 4.	1st Ave. Shaft House, 3 and 4.	1st Ave. Shaft House, 1 and 2.	11th Ave. Shaft.	Weehawken Shaft.
11 000-volt feeders.	Two 25-cycle.	Two 60-cycle.	Two 60-cycle.	Two 60-cycle.	Two 60-cycle.	Two 60-cycle.
Transformers	Nine 125-kw.	Two 125-kw.	One 125-kw. One 75-kw.	One 125-kw. One 75-kw.	Two 75-kw.	Two 75-kw.
Motor Generators.	Three 250-kw.	....	....	....	....	....
Horse Power of motors supplied	250	206.5	105	110	87.5	140
Switch-board panels	34	4	2	2	2	2
Arc lamps supplied.	50	....	....	....	....	....
Approximate number of incandescent lamps	1 100	1 100	625	625	300	800
Battery charging outlets	450	....	....	....	....	....
Voltage of motors	220	440	440	440	440	440
Voltage of lamps	110	32	32	32	32	32

The car-battery charging switch-board in this plant is worthy of special mention. The placing of cars at fixed positions required that their lighting batteries should be charged at any point in the yard, covering thirty-four tracks, with a total trackage length of 7 miles. In order to reach any car, it was necessary to provide circuits from a central point to outlets approximately 70 ft. apart at each track, and to make provision in the sub-station for a controlling switch-board of 34 panels. On these are mounted 476 five-point switches, which connect through suitable resistances to the respective outlets at the

tracks for regulating the current supplied to each battery. On these panels means are also provided for indicating the charging current and voltage on each circuit. The board was specially designed to occupy the minimum space, the 34 panels requiring a length of only 68 ft.

Auxiliary Sub-station "B."—Sub-station "B" is in the house over the shafts of Tunnels Nos. 3 and 4, at Long Island City. It is connected with the Power-House by two feeders. Step-down transformers are installed to supply power at 440 volts, 3-phase, to operate motors at the shaft sumps and at the Sunnyside Yard portals of the tunnels; also motors for operating the ventilating fans at this point. The same transformers, by a connection to the neutral point, supply 252-volt, single-phase current for lighting the tunnels. From this sub-station one 3-phase, 11 000-volt cable is run through each of the four tunnels to Auxiliary Sub-stations "C" and "D"; two to each sub-station.

Auxiliary Sub-stations "C" and "D."—Sub-station "C" is over the First Avenue shaft of Tunnels Nos. 3 and 4, and "D" is over the shaft of Tunnels Nos. 1 and 2. Two sub-stations were provided at this point because it was not feasible to connect the two shaft-houses by ducts and power cables, as was done at Long Island City. These sub-stations are similar to "B," except that there are no outgoing 11 000-volt cables. In each of these sub-stations is installed a step-down transformer for supplying alternating-current power to the fan and pump motors and for the tunnel lights.

Auxiliary Sub-stations "E" and "F."—Sub-stations "E" and "F" are at the Eleventh Avenue and Weehawken shafts, respectively, of the North River Section, and are supplied with 11 000-volt current by two cables from the Service Plant. These cables are tapped into Sub-station "E" at the Eleventh Avenue shaft, and terminate at Sub-station "F." Each sub-station is equipped with lowering transformers for operating motors for sump pumping, ventilating, and tunnel lighting.

Switching Stations.

Intermediate between the traction sub-stations are located switching stations, in chambers built in the tunnel system, and in small buildings adjacent to the line in the open. These stations are not provided with attendants; they contain switches for sectionalizing and cross-connecting the direct-current third-rail system. With two exceptions, the stations are equipped for the remote control of switches and circuit-

breakers, and are operated from the nearest traction sub-station by special control circuits, the two which are provided with hand-controlled switches are at or near signal cabins, and are operated by the signalman.

TABLE 11.—SWITCHING STATIONS (THIRD-RAIL SYSTEM).

No.	Location.	No. of main line feeders.	No. of yard feeders.	4 000-Ampere Circuit-Breakers: Hand-operated.	4 000-Ampere Circuit-Breakers: Remote-controlled.
1-A	Sunnyside Yard	10	7	6	10
2-A	Tunnel No. 3 west of First Avenue Shaft	4	..	..	4
2-B.	Tunnel No. 2 west of First Avenue Shaft	4	..	..	4
3-A	East of Weehawken Shaft	4	..	..	4
3-B	Secaucus	3	..	..	2
4-A	Hackensack River	5	..	5	..
4-B.	Signal Cabin "N", Manhattan Transfer	4	1	5	..

Distribution and Control.

Distributing Circuits.—In general, the direct-current traction feeder circuits consist only of the third-rail and track return, with short cable connections between the sub-stations and the third-rails and tracks, these being sufficient to carry the currents without excessive losses. The only exception is between Sub-stations Nos. 3 and 4, where the distance is such that it is necessary to supplement the third-rail and track return by a positive and negative feeder; these are each of 2 000 000 cir. mils section, and are carried on the high-tension pole line. The return feeder is continuous between sub-stations, with a connection to each track at each automatic signal. The positive feeder is omitted between switching stations.

It is necessary, in order to avoid undue current loss or excessive first cost, to connect all sub-stations in parallel through the third-rail, and to interconnect the third-rails of the two tracks at intermediate points between sub-stations. An accident, such as the grounding of the third-rail, under these conditions, however, would ordinarily affect the entire section of line; in order, therefore, to secure the advantages of the interconnected system, and at the same time to permit of ready isolation of any section of track without interfering with operation on the adjacent tracks, a system of sectionalizing was installed. To do this, the switching stations above refered to have been provided, in

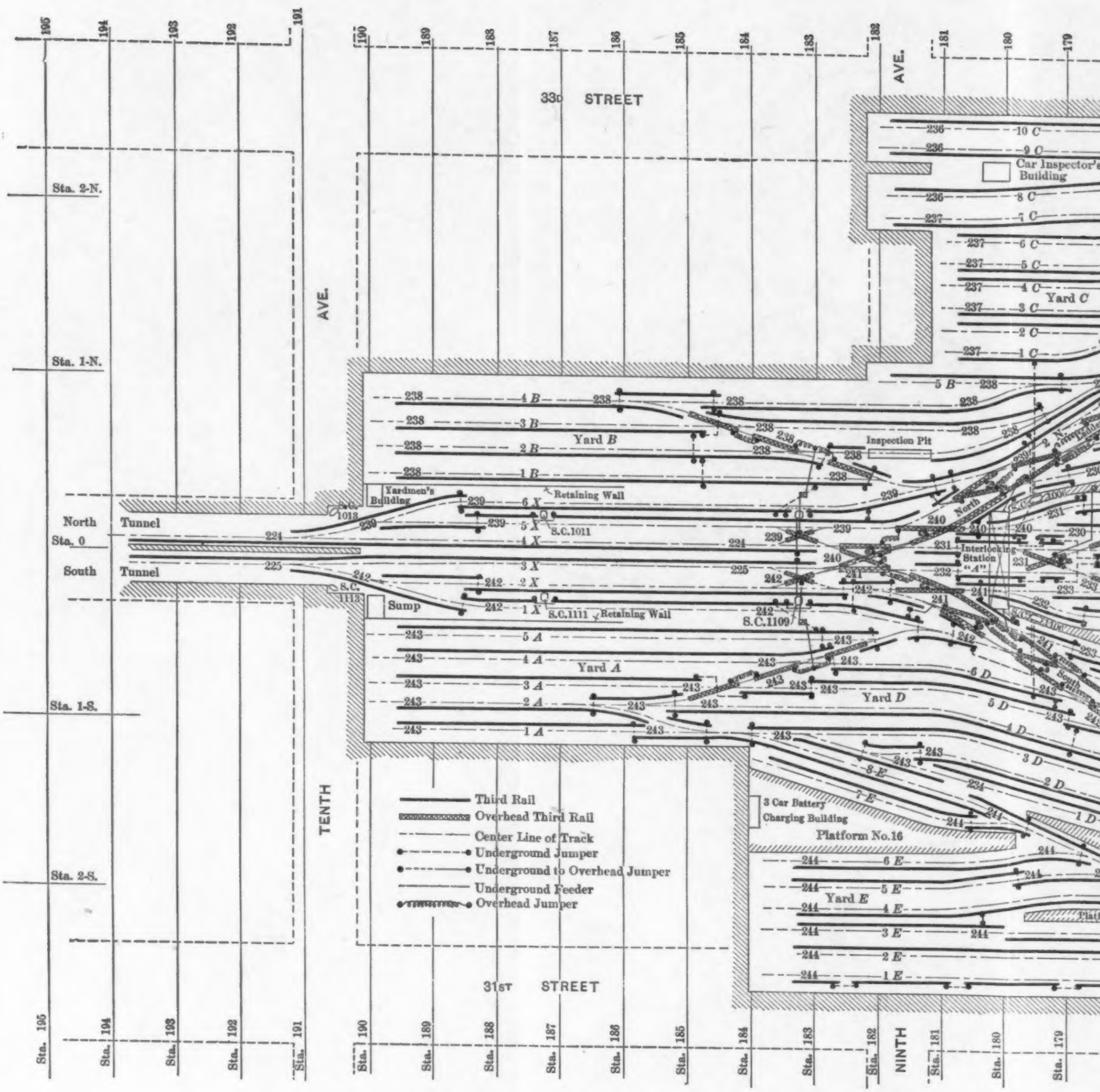
addition to the normal switching apparatus in the sub-stations; and the connections at the switching stations are such that by remote control the operator in any sub-station has complete control of each section of track between that sub-station and the nearest switching station. The circuit-breakers are also automatic at both points, so that on over-load or short-circuit the section affected is cut out only between the sub-station and the nearest switching station, and on one track only.

To restore the section after the over-load, the operator closes a switch in the sub-station, and this, through the remote-control circuit, actuates the switch in the switching station. At the hand-operated switching stations, the switches are closed by the signalmen on receiving instructions from the sub-stations.

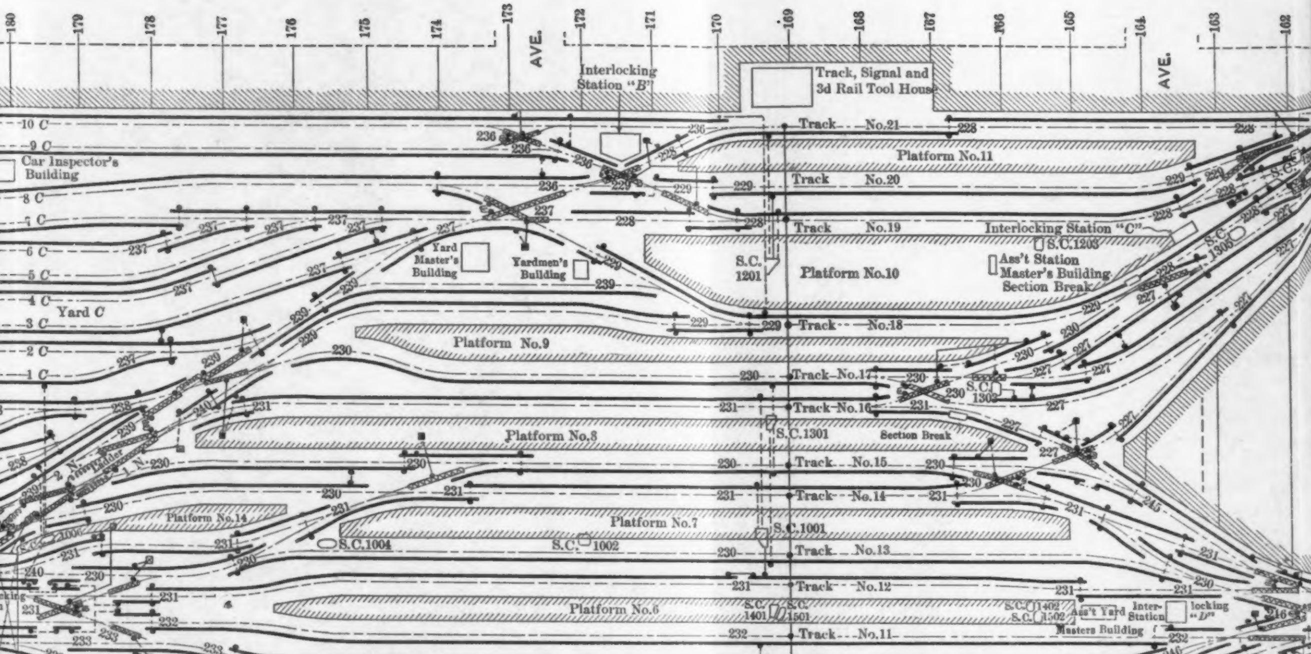
At the yards a complete system of sectioning is also provided. This is especially necessary because of the multiplicity of tracks, and the necessity for isolating a limited section only, which may be affected by the accident. In the station yard the tracks are divided into twenty-one sections, as shown on Plate XCVIII, and these are planned so that an accident on any one section will not interfere with through movements on the main tracks, and will not isolate an undue portion of the station tracks. The station yard sectioning is effected by switches in the Service Plant Sub-station, and under the control of the operator there. Similarly, the third-rails of the Sunnyside Yard tracks are divided into seven sections, controlled in the yard switching station.

Third-Rail Sectioning Switches.—In addition to the provisions for sectioning and cross-connecting the third-rails, as above described, this rail in each tunnel is sub-divided into sections, each about 1 500 ft. in length, by quick-break knife-switches. These are located approximately at each of the signals, and where the signals are far apart they are placed half way between. They are normally closed, but may be opened in emergencies, such as wreck or derailment, or the grounding of the third-rail, so as to localize the trouble and allow trains not immediately within this section to be operated out of the tunnel, and also to allow electric wrecking trains to be operated close to the point of derailment to assist in clearing up the wreck, or to haul stalled equipment from the tunnels.

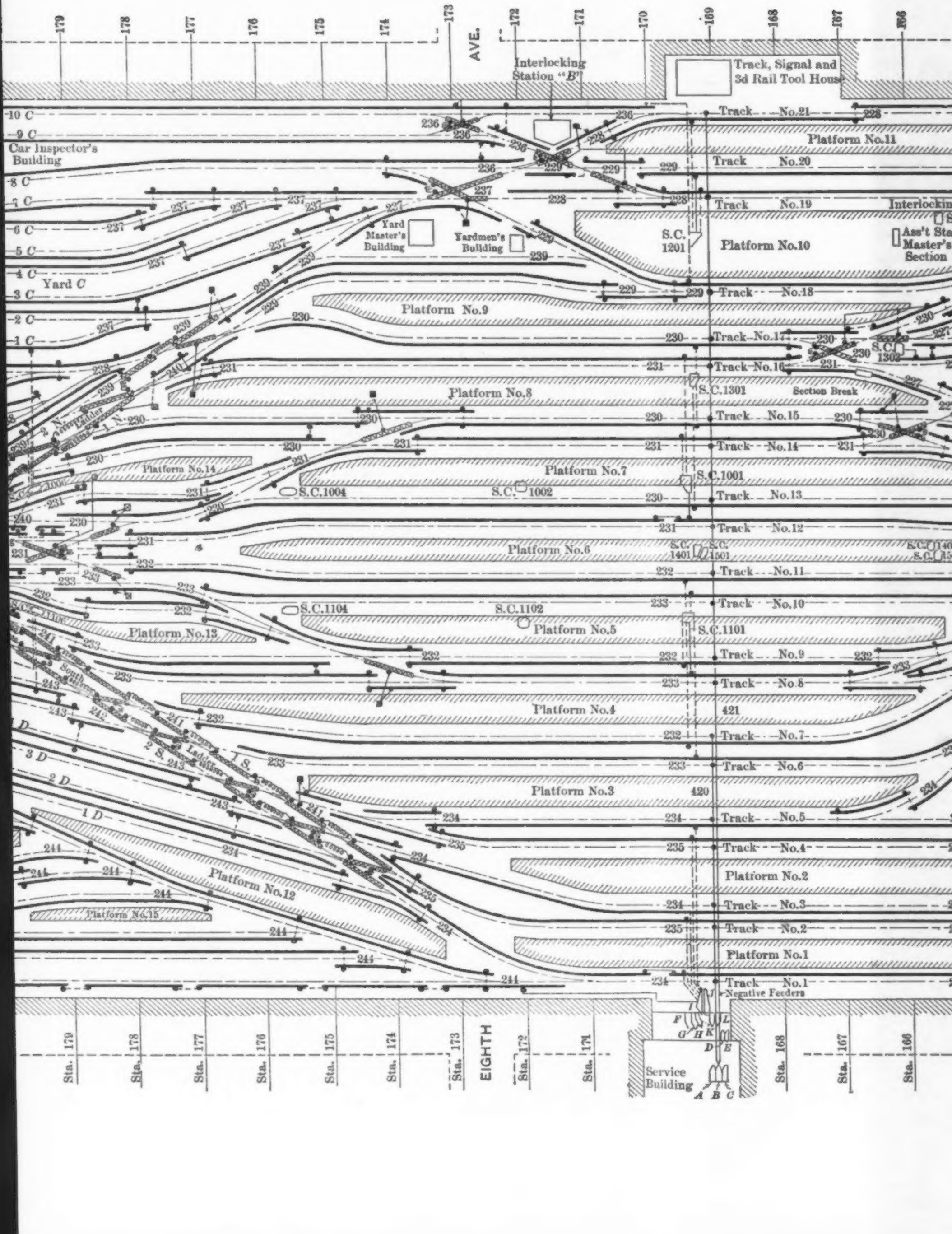
"Power Off" Signals.—In connection with the sub-division of the third-rail into sections controlled from sub-stations and switching sta-



33D STREET
31ST STREET
TENTH AVE.
NINTH AVE.
Sta. 2-N.
Sta. 1-N.
Sta. 0
Sta. 1-S.
Sta. 2-S.
North Tunnel
South Tunnel
Yard A
Yard B
Yard C
Yard D
Yard E
Car Inspector's Building
Yardmen's Building
Retaining Wall
Sump
Inspection Pit
Interlocking Station "A"
3 Car Battery Charging Building
Platform No.16
S.C.1011
S.C.1111
S.C.1109
S.C. 1013
S.C. 1113
Third Rail
Overhead Third Rail
Center Line of Track
Underground Jumper
Underground to Overhead Jumper
Underground Feeder
Overhead Jumper
Sta. 195
Sta. 194
Sta. 193
Sta. 192
Sta. 191
Sta. 190
Sta. 189
Sta. 188
Sta. 187
Sta. 186
Sta. 185
Sta. 184
Sta. 183
Sta. 182
Sta. 181
Sta. 180
Sta. 179



AVE.
Interlocking Station "B"
Track, Signal and 3d Rail Tool House
Car Inspector's Building
Yard C
Yard Master's Building
Yardmen's Building
Interlocking Station "C"
Ass't Station Master's Building
Section Break
Ass't Yard Masters Building
Interlocking Station "D"
Platform No.1
Platform No.2
Platform No.3
Platform No.4
Platform No.5
Platform No.6
Platform No.7
Platform No.8
Platform No.9
Platform No.10
Platform No.11
Platform No.12
Platform No.13
Platform No.14
Platform No.15
Track No.1
Track No.2
Track No.3
Track No.4
Track No.5
Track No.6
Track No.7
Track No.8
Track No.9
Track No.10
Track No.11
Track No.12
Track No.13
Track No.14
Track No.15
Track No.16
Track No.17
Track No.18
Track No.19
Track No.20
Track No.21
S.C.1001
S.C.1002
S.C.1004
S.C.1101
S.C.1102
S.C.1104
S.C.1201
S.C.1203
S.C.1301
S.C.1303
S.C.1305
S.C.1401
S.C.1402
S.C.1501
S.C.1502
Negative Feeders
Service Building
North Ladder
South Ladder
EIGHTH
SEVENTH
Sta. 180
Sta. 179
Sta. 178
Sta. 177
Sta. 176
Sta. 175
Sta. 174
Sta. 173
Sta. 172
Sta. 171
Sta. 168
Sta. 167
Sta. 166
Sta. 165
Sta. 164
Sta. 163
Sta. 162



179
178
177
176
175
174
173
AVE.
172
171
170
169
168
167
166
Interlocking Station "B"
Track, Signal and 3d Rail Tool House
Car Inspector's Building
Yard C
Yard Master's Building
Yardmen's Building
S.C. 1201
Ass't Station Master's Section
Track No.21
Platform No.11
Track No.20
Track No.19
Platform No.10
Track No.18
Platform No.9
Track No.17
Track No.16
S.C. 1301
Section Break
Platform No.8
Track No.15
Track No.14
Platform No.7
S.C. 1001
S.C. 1004
S.C. 1002
Platform No.14
Track No.13
Track No.12
Platform No.6
S.C. 1401
S.C. 1501
Track No.11
Track No.10
S.C. 1104
S.C. 1102
Platform No.5
S.C. 1101
Platform No.13
Track No.9
Track No.8
Platform No.4
421
Track No.7
Track No.6
Platform No.3
420
Track No.5
Track No.4
Platform No.2
Track No.3
Track No.2
Platform No.1
Platform No.12
Platform No.15
Track No.1
Negative Feeders
Service Building
Sta. 179
Sta. 178
Sta. 177
Sta. 176
Sta. 175
Sta. 174
Sta. 173
EIGHTH
Sta. 172
Sta. 171
Sta. 168
Sta. 167
Sta. 166

PLATE XCVIII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.

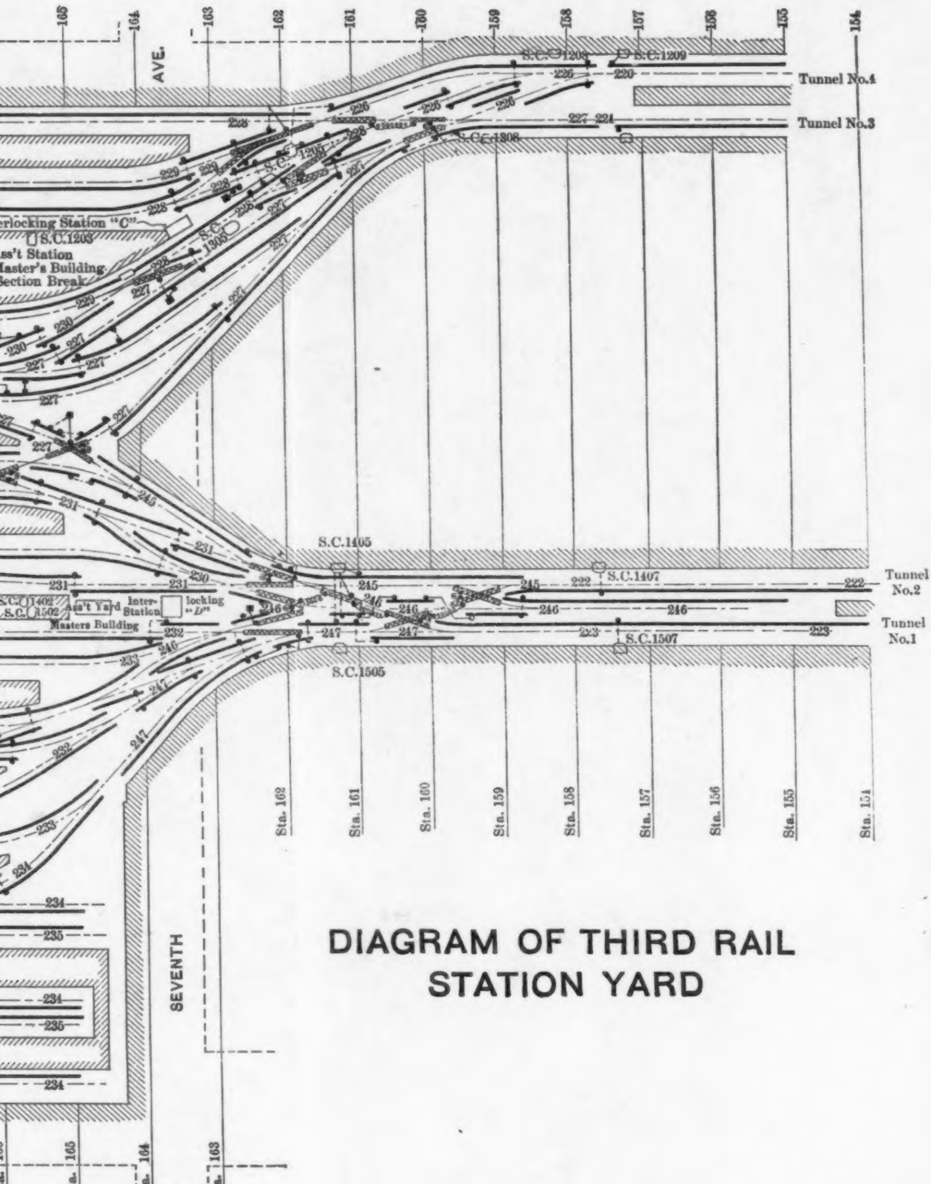


DIAGRAM OF THIRD RAIL STATION YARD

tions, provisions have been made to prevent a train from running on a dead section of third-rail. To do this, the rail is sectioned at certain of the automatic signals, and a relay is provided, with connections to the third-rail and to the signal, to which the circuits are arranged so that when the rail is dead the signal indication will be "Stop," and at the same time a gong will ring at the signal to notify the motorman, when he brings his train to a stop, that he is not to proceed. Where the sections occur at interlockings, the indication is given to the operator in the signal cabin.

Tunnel Alarm System.—Certain conditions may occur when it may be desirable to cut off the current from the third-rail because of a high-resistance ground or a short-circuit which does not pass sufficient current to actuate the automatic circuit-breakers in the sub-stations and switching stations, or when there may be danger of injury to workmen or others by accidental contact with the third-rail. In such emergencies the tunnel alarm system permits of immediately making the third-rail dead. The details of this system are described in the section devoted to "Tunnels."

Third-Rail and Track Return.

The Terminal operation, as elsewhere explained, required through service with the existing traction system of the Long Island Railroad, comprising more than 100 miles of electrified track. The top-contact type of third-rail is in use on this latter road, and has been found satisfactory, the cost of maintenance being very low, and it has the important advantage of simplicity of parts, flexibility, ease of maintenance and installation, and is easily repaired in cases of derailment. Third-rail location and clearances have been standardized by the American Railway Association, and, as a matter of course, the Long Island and the Terminal Railroad installations have been made to conform therewith.

Rail Section.—On account of the very heavy currents used for individual trains, and the density of traffic, it is necessary to have large current-carrying capacity and conductivity in the collector system. The capacity is in part fixed by the distance between the feeding points, and the required conductivity may be provided either by a light third-rail, supplemented by copper feeders, or a rail of large cross-section and special composition, which in most cases would require no supplemental feeders. The latter arrangement was adopted as being more economical

in first cost and of greater simplicity. The rail section used on the main-line tracks is the heaviest yet adopted for traction purposes, being 150 lb. per yd., and of special chemical composition, low in carbon and other hardeners, giving a resistance of about eight times that of copper, instead of about twelve times, as in the ordinary track rail.

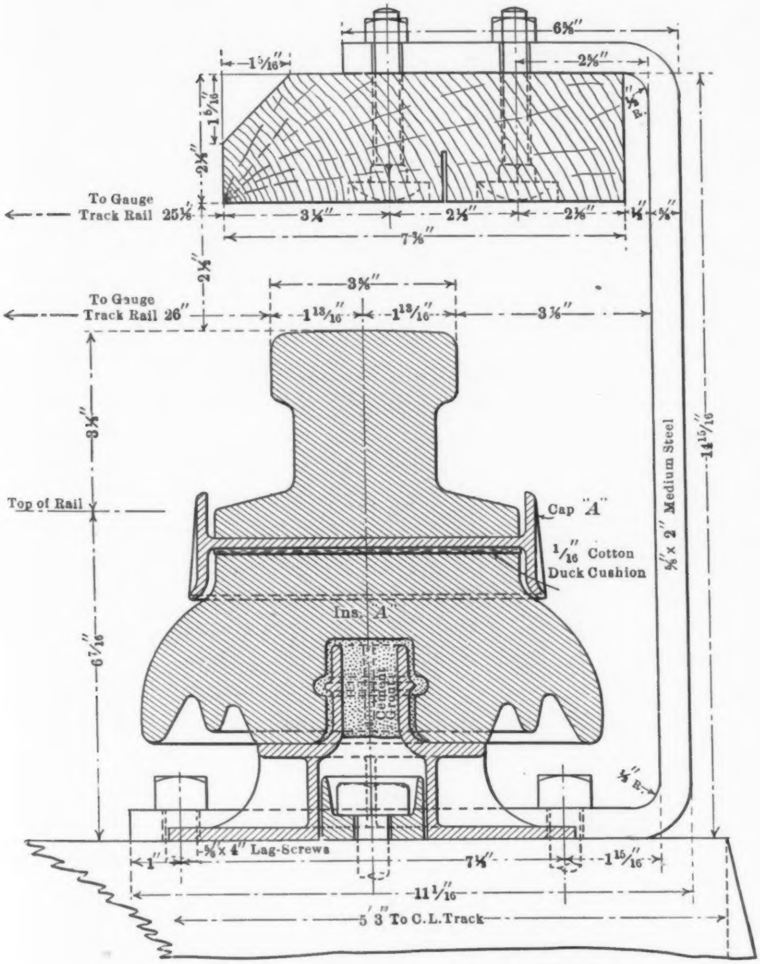
The rail, Fig. 16, is a **T** section having a vertical height of 4 in. and a lower flange width of 6 in., making a section which allows for ample insulation from the track ties, and is stable against overturning; it also provides for a simple form of splice, and convenient bonding at the joints. In the yards, where the large section is not needed for conductivity, and where it is desirable to have the maximum of clearance for signal and other apparatus along the track, the section consists of 25-lb. standard Bessemer **T**-rails, Fig. 17, mounted in an inverted position, the foot of the rail constituting the contact surface.

Insulators.—Experience on the Long Island Railroad and elsewhere, with various types of insulators, has resulted in the adoption, by the Terminal Railroad, of a simple and substantial form of insulating block for supporting the third-rail. These blocks are of porcelain, made by the "dry" process, which gives a very tough and mechanically strong insulator, with ample and permanent insulating qualities. For the open sections of the road a simple rectangular block with rounded corners is used; in the tunnels, where in places there is dampness due to condensation, or salt seepage-water, a petticoat-type insulator is used, which furnishes a more extended surface to provide against leakage of current.

Bonding.—The heavy-section rail is bonded with ribbon-type, compressed, terminal foot-bonds, four to a joint, having a conductivity equivalent to 80% of that of the third-rail. The light-section rails for the yards are bonded with the protected-type, pin-terminal cable-bonds.

Protection.—The third-rails throughout are protected by a continuous plank carried on wrought-iron brackets secured to the third-rail ties, as shown by Figs. 16 and 17. On the open line, this plank is of yellow pine, but, in the tunnel, Jarrah wood, imported from Australia, has been used, because of its slow-burning qualities.

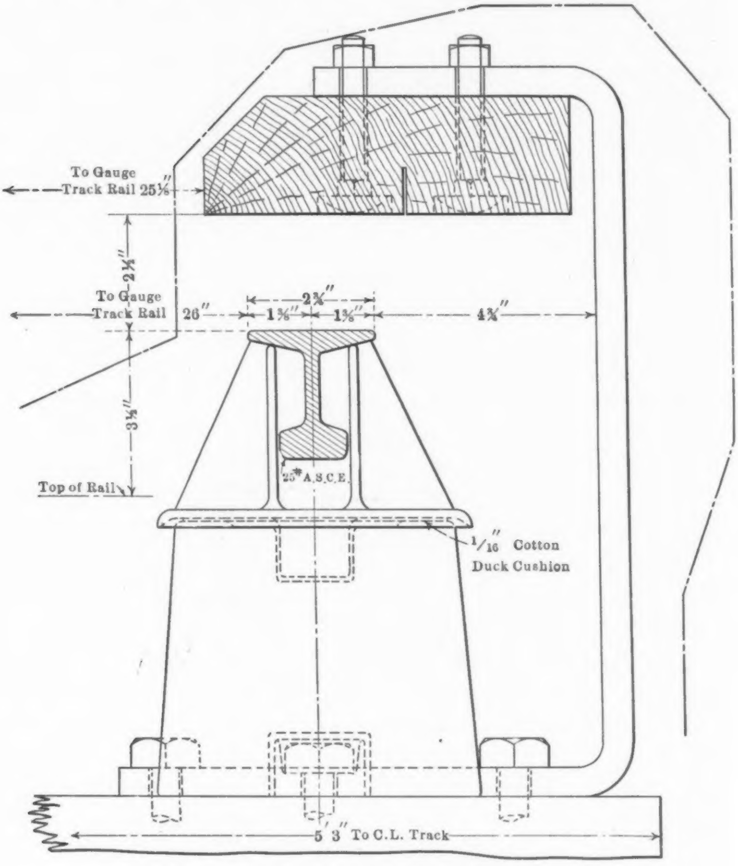
Connections.—The connections to the third-rail from sub- and switching stations are made by insulated cables of 2 000 000 cir. mils section for the heavy rail, and 1 000 000 cir. mils for the light rail. The cables terminate in special porcelain "pot-heads" from which flexible cables are connected to the third-rail by bond terminals.



SECTION OF THIRD RAIL AND PROTECTION
FOR USE IN TUNNELS.

FIG. 16.

Track Bonds.—Throughout the main tracks, both rails of each track are bonded; in the yards, however, as elsewhere explained, only one rail is bonded. In the open, where there is danger of theft of copper, the bonds are of the protected type, placed under the joint angle-bars; in



SECTION OF THIRD RAIL AND PROTECTION FOR YARD TRACKS.

FIG. 17.

the tunnels and terminal yard, from which the public is excluded, long cable bonds are run around the angle-bars. All bonds are of wire cable, provided with forged copper terminals, and are secured in drilled holes in the rails by steel pins driven through the center of the heads. As a

matter of interest, it will be noted that there was great difficulty in drilling the open-hearth steel rails for these connections; it was found that tools suitable for Bessemer rails would drill only from two to four holes in the open-hearth rails; therefore special steels had to be used for the drill bits.

TABLE 12.—RAIL BONDS.

Weight of rail.	Rail.	Type of terminal.	Application.	Bonds, per joint.	Circular mils, over bond.	Length of bond, in inches.
150-lb........	Third......	Compressed...	Exposed.......	4	450 000	5 and 10
100-lb	Track......	Pin............	Protected	2	273 397	20 and 25
100-lb........	"	"	Exposed.......	2	400 000	49 and 53.25
100-lb........	"	"	Protected	2	300 000	25
85-lb	"	"	"	2	300 000	15
25-lb........	Third......	"	"	2	0000 B. & S.	12
25-lb	"	"	Exposed.......	2	00 B. & S.	24.25

Overhead Third-Rail.—In order to provide continuous power supply to electric locomotives passing through certain cross-overs and the ladder tracks of the Station yard, it was found necessary to supplement the usual ground third-rail construction by sections of overhead contact; the arrangement, furthermore, permits of the simplification of the contact system in the complicated special work of the tracks, reduces the number of cable cross-connections, and allows more space for the installation and care of switch-operating apparatus. In general, it has been designed to give continuous contact through all ladders and for important locomotive switching movements, but, for minor movements and single or two-car multiple-unit operation it has been assumed that coasting over gaps in the ground third-rail will be permissible.

The overhead contact consists of a 25-lb., standard-section T-rail, hung flange downward through insulators from longitudinal lattice box-girders. Under the buildings and viaducts these girders are supported by the floor structure, and in the open yard they are carried by cross-girders supported by columns between tracks. To reduce the number of columns, the cross-girders are designed in part as simple spans, and in part as cantilevers, and the longitudinal system as cantilever trusses having curved top chords. The structure carries generally two parallel lines of third-rail, one for each of the tracks of the double ladder systems; and in many locations it is used as a support for the interlocking and block signals. Reinforced concrete

fenders have been placed at the approach in either direction about the lower portions of all columns, as a protection against damage from derailment of rolling stock.

The locomotives are equipped with air-operated, overhead, pantagraph shoes, which may be raised by the motorman when desired, to make contact with the rail, and the contact surface of the latter is sheathed with heavy sheet-copper, in order to reduce sparking when heavy currents are thus drawn. Multiple-unit motor cars have not been provided with top contact shoes. No overhead contact system has been found necessary in the Sunnyside or Manhattan Transfer Yards.

The following are general data relating to the Station yard overhead system:

Total length of contact system..............	3 930 ft.
Total " " " surface..............	6 210 "
Number of sections of rail..................	86
Distance from top of track rail to contact surface	15 ft. 4 in.
Distance from top of track to bottom chord of structure	16 ft. 7 in.
Length of complete cantilever span..........	122 ft.
Depth of cantilever at supports.............	6 "
Depth of cantilever at middle...............	3 "
Width of cantilever.........................	3 "
Number of columns required for entire system.	12

Conduit System.

Throughout the tunnels and terminal area all cables for power, and telephone and telegraph service, are run in a permanent system of tile conduits, described in detail as to their constructive features in other papers. There are in all approximately 400 duct miles of power conduits and 600 duct miles of telephone and telegraph conduits. The general scheme involved in their planning is as follows:

Power Conduits.—The system of power conduits originates at the Long Island City Power-House, from which two separate banks of tile ducts are run to the four water-front tunnel shafts. At these points vertical pipe conduits are embedded in the shaft linings and connect to the splicing chambers of the ducts in the tunnel benches. The ducts extend in one bench of each tunnel eastward to the portals in Long Island City and westward to the throat of the yard, near Seventh Avenue. From this point they are continued westward below the

yard sub-grade to a point opposite the Service Plant between Seventh and Eighth Avenues; thence southward to the splicing chambers in the basement of this building. The arrangement of ducts is similar through the yard to the North River Tunnels and through them to the Hackensack Portal; thence they run along the walls of the approach and into Sub-station No. 3, where the duct lines terminate. The conduit system above described furnishes housing for all classes of high-tension cables, and permits of dividing the cables into groups, so that independent duplicate routes are provided to each power-transforming point.

Telephone and Telegraph Conduits.—A similar system has been provided for other than power purposes, such as telephone and telegraph. In the tunnels these ducts occupy the opposite bench from the power ducts, and in the terminal yard they consist of separate lines. This telephone conduit system was designed for the use of the Railroad Company, but connections have been made at various points to other telephone and telegraph systems, in Long Island City, in Manhattan, and in New Jersey. In Long Island City the conduits terminate at the tunnel portals, but at the water-front certain ducts have been brought to the surface at the First Avenue shaft and others at Sixth Avenue, 32d and 33d Streets. In the terminal yard a number of ducts terminate at a distributing chamber under the Station Building, and others at Ninth Avenue. At the Hackensack Portal all ducts terminate, and connection is here made with the pole lines of the Postal Telegraph Cable Company and the Railroad Company.

It will be noted, from Table 13, that the number of conduits in the various sections is not the same; this was caused by the advisability of installing in the permanent tunnel construction as many ducts as there was convenient space for, the limit being the maximum number of cables which could be cared for properly in the splicing chambers. Obviously, therefore, there are fewer ducts in the river tunnels, where the size of the splicing chambers is limited, than in the land tunnels and the Station area.

Pole Lines.

The Meadows section of the railroad is a 5-mile continuous stretch of semi-tidal meadow swamp land, except for a short section of rock outcropping at Snake Hill. The Hackensack River is crossed midway of the section. The ground surface is covered with a heavy growth

of reeds, and the top stratum is a peaty bog, from 8 to 15 ft. deep, underlaid with varying strata of clay, fine sand, and mixed sand and clay for very considerable depths. Across this section, and adjoining the track embankment, a pole line was erected for telegraph and telephone purposes, Fig. 1, Plate XCIX, and one for the high-tension power wires, Fig. 2, Plate XCIX.

TABLE 13.—Cable Conduit System.

Location.	From:	To:	No. of Power Ducts.	No. of Telephone and Telegraph Ducts.	Total.
North River Tunnels............	Hackensack Portal	Weehawken Shaft	North Tunnel 24	North Tunnel 48	72
			South " 24	South " 48	72
	Weehawken Shaft	11th Avenue Shaft	North Tunnel 15	North Tunnel 40	55
			South " 15	South " 40	55
	11th Avenue Shaft	10th Avenue Portal	North Tunnel 36	North Tunnel 20	56
			South " 36	South " 20	56
	10th Avenue Portal	Terminal Manhole	North Side 36	North Side 21	57
			South " 36	South " 21	57
Terminal Area..	Center of Terminal	Service Plant	184	Pipes from Terminal Room	
	Center of Terminal	6th Avenue	Tunnel No. 1-30	Pipes in Overhead Gallery	
			" " 2-30		
			" " 3-30		
			" " 4-30		
East River Tunnels.............	6th Avenue	1st Avenue Shaft	Tunnel No. 1-30	Tunnel No. 1-48	78
			" " 2-30	" " 2-48	78
			" " 3-30	" " 3-48	78
			" " 4-30	" " 4-48	78
	1st Avenue Shaft	Long Island Shaft	Tunnel No. 1-15	Tunnel No. 1-40	55
			" " 2-15	" " 2-40	55
			" " 3-15	" " 3-40	55
			" " 4-15	" " 4-40	55
	Long Island Shaft	Long Island Portals	Tunnel No. 1-15	Tunnel No. 1-24	39
			" " 2-15	" " 2-24	39
			" " 3-15	" " 3-24	39
			" " 4-15	" " 4-24	39
Long Island City.........	Long Island Shaft	Long Island Power-House	40	....	....

Telegraph Line Poles.—Ultimately, the telegraph and telephone service will require 60 open wires and two 40-pair cables, and it was desired to make this line entirely secure against probable interruption by severe storms or fires in the swamp reeds. The character of the foundation, as indicated, was bad, and, after much consideration, it was decided to substitute for an H wooden pole line, which would be inadequate for the conditions, one of concrete poles, which, while somewhat experimental, and perhaps somewhat more costly, would provide a safe and durable construction.

TABLE 14.—POLE LINES.

Description.	Length of line, in miles.	Number of poles.	Length of spans, in feet.	Heights of poles, in feet.	Present installation.	Total future capacity.
Manhattan Transfer Yard: Power-transmission line..........	2.18	40	300	32 to 70	1 ground wire... 4 signal wires....	1 ground wire. 6 power wires. 4 signal wires.
Meadows Section: Power-transmission line....................	4.60	77	300	50 to 70	1 ground wire...... 9 power wires...... 4 signal wires ..... 2 feeder wires.....	1 ground wire. 24 power wires. 4 signal wires. 2 feeder wires.
Meadows Section: Concrete telegraph and telephone poles ...	4.60	202	120	35 to 65	34 open wires. ..... 1 telephone cable.. 1 signal cable......	60 open wires. 2 telephone cables. 1 signal cable.
Sunnyside Yard: Signal pole line..............................	0.72	23	150	22 to 45	Varies from 2 to 8 wires.......	2 signal wires. 8 signal wires. 1 signal cable.
Sunnyside Yard: Power and lighting lines....................	0.13	67	150	34 to 50	Maximum.	1 ground wire. 6 power wires. 2 fire-alarm wires. 4 arc-light wires.

In this section 202 poles were required. They were spaced from 70 to 135 ft. apart, with an average standard span of 120 ft., the variations in span being due to the numerous railway and highway crossings. The heights of the poles above the ground vary from 25 to 50 ft., and they are from 35 to 65 ft. in total length.

The design, made by R. D. Coombs, M. Am. Soc. C. E., Structural Engineer on the staff of the writer, called for transverse loading conditions, in case of maximum storms, equivalent to 6 000 lb. at 6.5 ft. below the top of the pole for the 120-ft. span length. The poles are square in cross-section, with chamfered corners and with a taper of ½ in. in 5 ft. The 1:2:4 concrete mixture of which they are made was assumed to have an ultimate unit strength, in compression, of 2 200 lb. The reinforcement is composed of mechanical bond bars tied together into a square skeleton frame. In the completed pole, this reinforcement is covered by a 1-in. minimum thickness of concrete. The skeleton reinforcement was placed in horizontal frames, and the concrete mixture was poured in and carefully tamped. A special yard (Fig. 1, Plate CI) was established near the line, in which to make, store, and season the poles. The average number of poles made per day was six, and they were left in place 16 days to season.

After a number of experiments, it was found best to set the poles in pits (Fig. 1, Plate C) excavated in the marshy stratum. These pits were generally about 9 ft. square and 5 ft. deep, and a timber grillage was placed around the base of each pole, and about 5 ft. below the top of the ground. This grillage consisted of six track cross-ties bolted together and to the pole, and partly planked over by 3-in. rough lumber. The pole, which projected below the grillage and was pointed at the butt, was jetted down by compressed air into the sandy layer, so that the grillage would rest at the bottom of the pit. The pits were then back-filled with rock and clay. Poles on curves are cross-guyed, and the terminal and railway crossing poles are head-guyed with steel cables. Fig. 2, Plate C, shows a pole 55 ft. long, with 13 ft. penetration, and tested by an ultimate load of 4 360 lb. applied horizontally 39.5 ft. from the ground. Subsequent poles were of modified design, giving greater strength.

Because of the unusually heavy line and the extra length required for the foundations, the gross weight per pole, exclusive of grillage

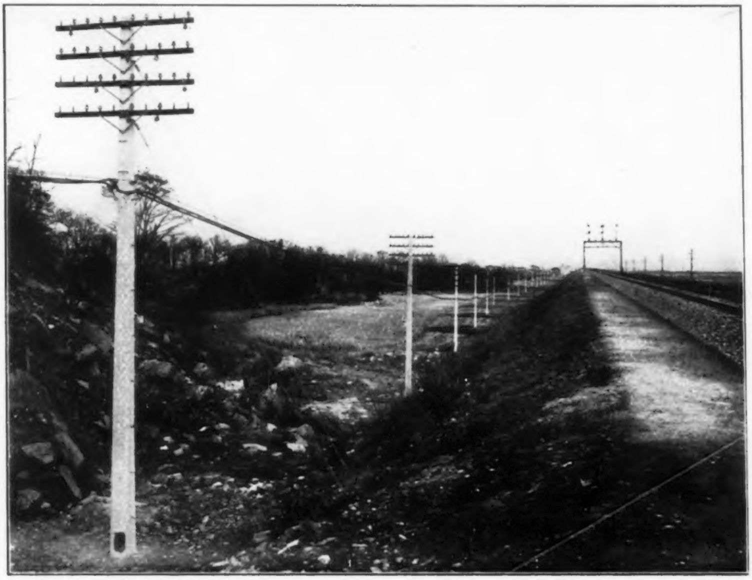


FIG. 1.—CONCRETE POLE TELEGRAPH LINE, MEADOWS DIVISION.

FIG. 2.—POWER TRANSMISSION LINE, MEADOWS DIVISION.

and cross-arms is more than would be required for ordinary telegraph poles, and varies from 5 300 lb. for a 35-ft. pole to 17 300 lb. for poles 65 ft. in length.

Transmission Line Poles.—As elsewhere described, the wires for the transmission of traction power from the tunnel portal to Harrison Sub-station, and the wires of the high-tension signal power circuits in the same section, are carried on a line of steel poles along the southern edge of the right of way across the Meadows. These poles are set 300 ft. apart, and are designed for, not only the present requirements, but to carry seven additional three-wire transmission circuits which may be required in the future. The total loading called for a very substantial pole construction, and also for foundations to be carried through the soft upper strata of the marsh to a firm bearing.

The poles are of latticed structural steel, square in cross-section, with one angle at each corner and single-angle bracing. The poles have a parabolic outline, conforming to the load requirements and giving an improved appearance. The parabola is of such flat outline that it was not necessary to bend the main angles before assembling. The poles were completely riveted at the shop, with the exception of the cross-arms. The latter consist of single ship-channels with flanges turned downward. The pole has a cast-iron cap at the top, and a section of pipe to carry a 250 000-cir. mil copper ground-wire, which also forms a part of the negative, or return circuit.

In crossing the Hackensack River it was determined to carry the wires overhead rather than by submarine cables, in order to preserve the integrity of the line against lightning disturbances, and to provide for the use of 33 000-volt transmission in the future. For this purpose it was necessary to carry all wires with the clearance specified by the War Department over navigable streams, and this required the use of two unusually high steel towers. The line approaches the river with 300-ft. spans on 50-ft. poles, rising to an intermediate 70-ft. pole, then sharply to the high towers, 181 ft. 4 in. above high water; the lowest wire in this crossing is 137 ft. 4 in. above high water. The wire span over the river has a length of 765 ft. The towers are of the same general outline as the poles, but of much heavier section and larger dimensions. They are 15 ft. square at the base and 3 ft. square at the top. The tops of the foundations are 6 ft. above high

water, and the total height from the water to the ground-wire is 195 ft. The towers are carried on twin-pier, reinforced concrete foundations, each having eleven timber piles under it.

The pole foundations across the Meadows are of concrete on from eight to ten piles, depending on the size of the pole; the piles were driven to a depth of from 30 to 80 ft., as occasion required.

Although the character of the sub-surface strata on the Meadows had been studied from test borings, it was found that the conditions varied radically, and it was not possible, even after driving the piles for one foundation, to determine in advance the approximate length of those for the next (300 ft. distant). This uncertainty added materially to the difficulty of distributing the piles. The cut-off was below the level of the water in the marsh, and, to accomplish this, as well as to place the concrete foundations for the poles in the water-bearing ground, it was necessary to use steel sheet-piling and remove the water with power pumps.

A concrete-mixing train was used for the foundation work; this plant consisted of three cars, the center one containing the mixer with the engine and boiler, and end cars carrying the sand and stone; these cars were provided with steam coils to heat the mixture, as the foundations had to be built in cold weather.

The poles, both of steel and of concrete, were erected with a standard, 75-ton, wrecking derrick, fitted with a special 90-ft. boom capable of lifting either the steel or concrete poles at a point 90 ft. from the center of the track. The concrete poles were lifted from the cars on which they were loaded, and placed on timber horses adjacent to the excavation where they were to be set; the cross-arms and grillage were then put in place, and the pole, thus equipped, was picked up at the top and lowered into the excavation. The steel poles were picked up from the embankment, where they had been unloaded, and lifted by the derrick vertically over the foundation and set in place, as shown by Fig. 2, Plate XCI.

Insulators.—All high-tension insulators are of porcelain, of the petticoat type. Straight-line insulators are made of three pieces, and strain insulators of two pieces. These insulators are mounted on cast-steel pins bolted to the steel channel cross-arms.

Anchoring Devices.—The transmission line poles supporting the spans crossing the various railroads in the Meadows section, and the

FIG. 1.—GRILLAGE FOUNDATION FOR CONCRETE POLE.

FIG. 2.—TEST OF CONCRETE POLE: POLE 55 FT. LONG; 13 FT. PENETRATION; ULTIMATE LOAD, 4 360 LB., APPLIED HORIZONTALLY 39.5 FT. FROM THE GROUND. SUBSEQUENT POLES WERE OF MODIFIED DESIGN, GIVING GREATER STRENGTH.

city streets in the Sunnyside Yard section, were provided with double cross-arms, strain insulators, and a dead-end clamping device which was designed to attach the power wires securely to the structure.

At the Hackensack River the power line rises sharply, in one span, to the top of the high towers, and required special insulating attachments. Each power wire, in passing over the steel cross-arms of the tower, is carried in a saddle supported by a nest of four standard line insulators. The saddle is provided with a special six-bolt clamp, and its wire groove is curved to prevent sharp bending.

The 2 000 000-cir. mil direct-current feeders, in addition to a similar clamping saddle, have an auxiliary butterfly clamp on each side, about 2.5 ft. from the saddle and attached thereto by adjustable rods.

The high-tension signal power circuits in the Manhattan Transfer Yard are carried on a line of latticed steel poles, extending from the Harrison Sub-station to the Passaic River, at Newark.

Signal Lines.—In addition to the present signal circuits through the yard, and feeding the New York Division at the Passaic River, the pole line is arranged to carry in the future two 3-wire, high-tension, power circuits.

Long Island Railroad Pole Lines.—In Sunnyside Yard the Long Island Railroad traction power line was re-located through the yard and rebuilt from the arrester-house at Dutch Kills Street, to Woodside Avenue, a distance of 8 510 ft. The poles are of the latticed steel type, with straight lines, conforming in design to the original pole line of which it is a part. The cross-arms are of yellow pine, resting on shelf angles, and the attachments to the arms are made by U-bolts to avoid boring holes in the timber.

In Sunnyside Yard the signal power circuits are carried from the tunnels, on a line of light steel poles, to a connection with the Long Island Railroad pole line near Thompson Avenue. These poles are composed of three main members, which are half cylinders of rolled high-carbon steel, fastened together transversely with ties and spreaders.

Two high-tension circuits are carried across the South Yard from the Long Island Railroad pole line to Auxiliary Sub-station "A." The arc-lighting system of the yard is placed on poles of similar design, which also carry the fire-alarm and yard-lighting circuits. Table 14 contains additional data as to all pole lines.

ELECTRIC LOCOMOTIVES.

Quite independently of the general character of the traction system, it was felt that considerable experimental investigation was needed to decide on the type of electric locomotive suitable for handling heavy main-line trains. Therefore a special "Locomotive Committee" was appointed by the President to investigate existing designs and develop, by experiment, or otherwise, a suitable locomotive for the exacting conditions of the proposed service. This Committee consisted of A. W. Gibbs, General Superintendent of Motive Power, Pennsylvania Lines East, D. F. Crawford, General Superintendent of Motive Power, Pennsylvania Lines West, A. S. Vogt, Mechanical Engineer, Pennsylvania Railroad, and the writer, as Chairman.

As a result of the work of this committee, two locomotives were designed and built at Altoona in 1905. They were put in service, and given a continuous trial on the Long Island Railroad, hauling freight and passenger trains, and the tests continued for many months. These locomotives are of the double-truck type; the trucks are linked together at their inner ends and the outside ends are used for attachment to the train drawbar. This arrangement of running gear has since been adopted by other roads for slow-speed electric locomotives. Each of the four axles carries an electric motor, one locomotive having geared and the other gearless motors.

These locomotives were designed for a maximum speed of about 45 miles per hour on a level, with a normal train, as at that time it was intended to confine the terminal electric operation to the haul through the tunnels from the west portal at Bergen Hill, New Jersey, to the yard at Long Island City. Practically all this portion of the line is on heavy grades and over a short distance only, and a slow speed would conduce to economy and would not be prohibitive as to time consumed. Subsequently, it was determined to extend the terminal run to Harrison, N. J., on the west, and possibly to Jamaica on the east, involving level stretches of about 10 miles at each end. Hence it became necessary to adapt the locomotives to the higher speed conditions normally obtaining in main-line operation. Accordingly, the two locomotives under test were modified electrically to permit of a maximum speed of about 65 miles an hour. Tests with these machines had demonstrated their hauling power and successful operation at the slow speed for which they were originally designed, but when speeded up it was

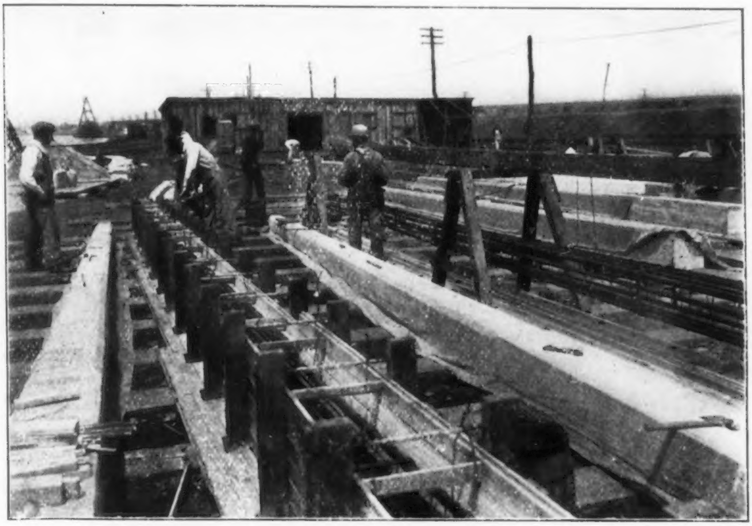


FIG. 1.—CONCRETE POLES IN PROCESS OF MANUFACTURE.

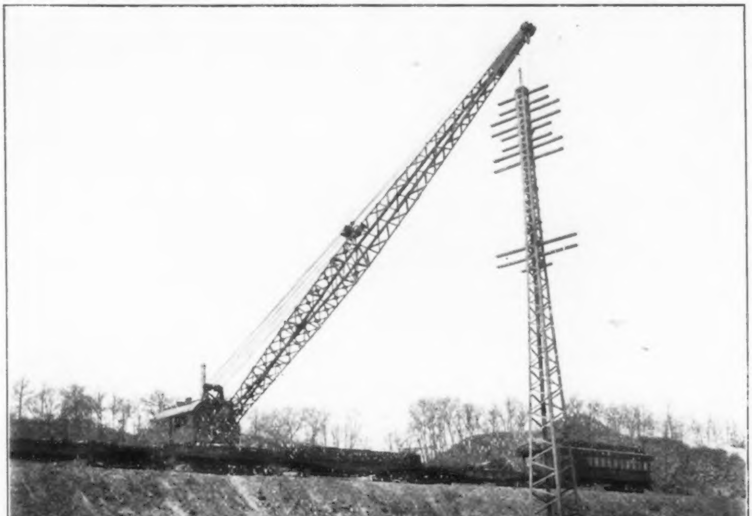
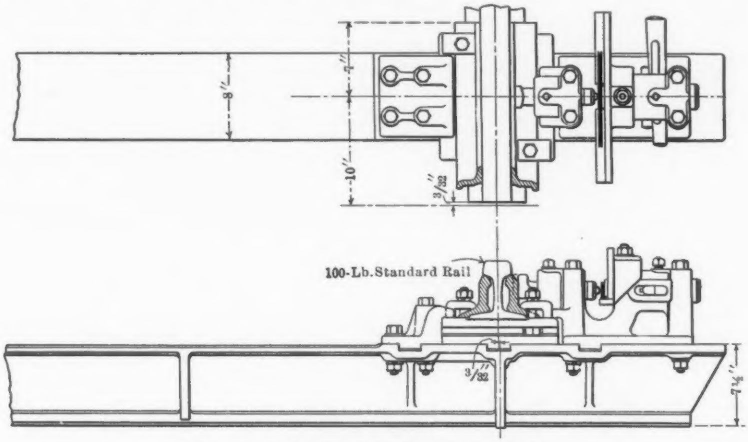


FIG. 2.—SPECIAL 90-FT. BOOM DERRICK ERECTING STEEL POLES ON MEADOWS DIVISION.

found that they became quite destructive to track. At speeds greater than 45 miles per hour they developed a tendency to rhythmic side swaying and the production of excessive lateral pressures at the rail heads. Such peculiarities in steam locomotives, with low-hung boilers, of course, were not altogether unknown to railway engineers, but they are intensified in an electric locomotive, where the power is applied with extreme compactness, and where a convenient method of motor attachment concentrates great weight around the axles; also, where it is not only possible, but generally most convenient, to utilize all weight for adhesion. Fig. 1, Plate CII, shows the motors and running-gear, and Fig. 2, Plate CII, is a view of an articulated electric locomotive.



SPECIAL TRACK TIE AND REGISTERING DEVICE FOR LOCOMOTIVE TESTS.

Fig. 18.

In order to bring out more fully the elements of design as affecting tracking, it was determined to institute a series of road tests, recording, as far as practicable, the comparative lateral rail pressures at various speeds with various types of steam and electric locomotives. A special recording apparatus (Fig. 18) for the purpose was devised and placed in a stretch of tangent track on the electrified portion of the West Jersey and Seashore Railroad. A complete series of tests was made, and, from the information obtained, the mechanical characteristics of the design of the locomotive to be built for tunnel operation were determined. During 1910 this test apparatus was again installed, this time in a section of the Terminal division track, and the new

locomotives were tested thereon to check their actual with their expected performance. The test was made over a length of 165 ft. of special track carried on cast-steel ties, having chairs near their ends for holding the rails. The chairs rest on rollers on seats in the ties, and allow for free lateral motion outward of the rails, except as restrained by the pressure-registering device on each tie. This device is in the form of a plug carried in a guide on the chair-seat casting; at one end it presses against the side of the rail head and at the other it carries a hardened steel ball which is placed in contact with a strip of plate steel. Proper adjustment for gauge is made before each run by wedges between the plugs and rails. Any side pressure at the rail from the wheel flanges of a locomotive moving over the track causes the steel

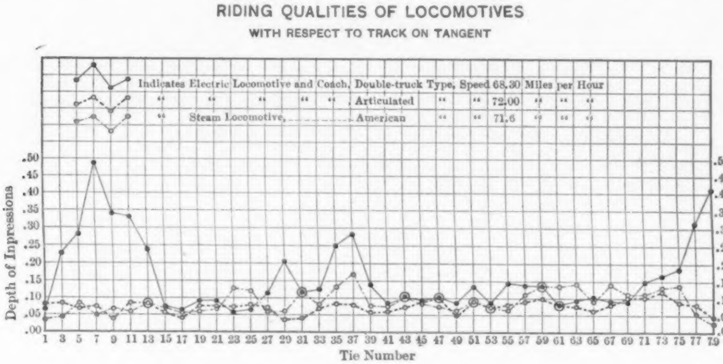


FIG. 19.

balls to press into and indent the steel-plate strips. The diameter of the impression, when measured by a micrometer microscope, indicates the magnitude of the side pressure, the location of the maxima, and the tendency of the locomotive to "nose" or oscillate. Typical samples of the records obtained along this track section are given in Fig. 19, one showing a normal record from a steam locomotive of the Atlantic type; another, from the original design of electric locomotive, and a third from the adopted type of electric locomotive. The tests were conducted to show the free-running characteristics of the locomotives at speeds as high as 94.6 miles per hour with a steam locomotive, and 86 miles per hour with an electric locomotive.

To test the pressure of individual wheels on curves, especially as affecting behavior in taking switches and turn-outs in yards, a hydraulic

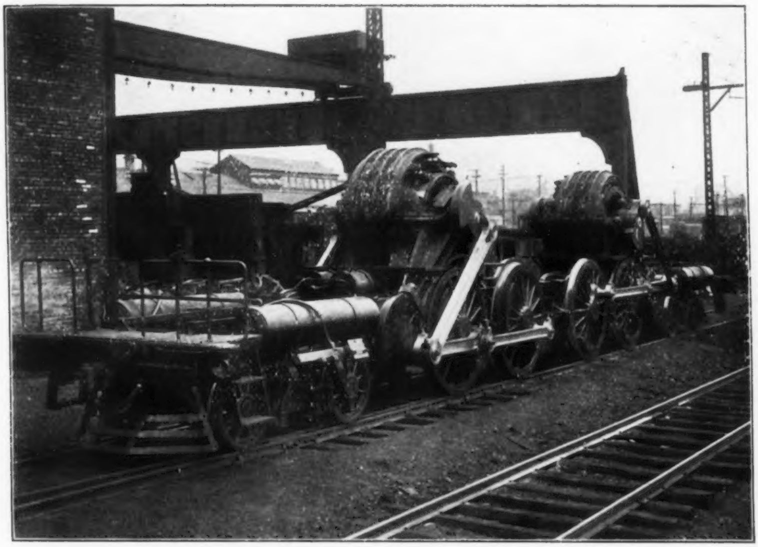


FIG. 1.—ELECTRIC LOCOMOTIVE, SHOWING MOTORS AND RUNNING GEAR.

FIG. 2.—ARTICULATED ELECTRIC LOCOMOTIVE.

apparatus devised by Mr. George L. Fowler was used. With this device the side pressure of each wheel flange is measured by connecting a short section of one rail through a system of levers to a hydraulic cylinder and its pressure-recording device. From experience obtained in the service tests of three different types of electric locomotives, and the results of the special track instrument tests, it was decided to make quite a radical departure from general practice in the final design of the high-speed locomotives for the terminal equipment. An attempt has been made to pattern the locomotive mechanically on the fundamental characteristics of modern steam locomotive design in the following particulars:

(*a*) High center of gravity of the machine as a whole, and especially of the heavy electric motor portion;
(*b*) The large proportion of the total weight spring-borne, and equalized by a system having considerable amplitude of motion;
(*c*) An unsymmetrical distribution of wheel-base of the locomotive;
(*d*) A combination of driving and carrying wheels.

DIAGRAM OF ELECTRIC LOCOMOTIVE.

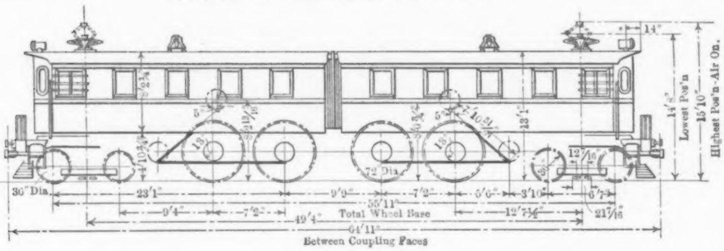


FIG. 20.

All the above characteristics, it was thought, would conduce to ease of riding, flexibility in tracking, and the reduction of destructive action to the roadbed by heavy masses moving at high speeds.

To accomplish these results, required an important modification in the customary method of mounting and connecting the electric motors; instead of being placed concentric with or in the plane of the axles, and direct-mounted or geared to them, they are placed on the main frames above the wheels, and driving connections are made with rods. The diagram, Fig. 20, and the photographs, Plate CII, show the general features of the design and the principal dimensions. The locomotive is double, or articulated, each half being similar to an "American" type,

or eight-wheeled steam locomotive, in the wheel arrangement, frames, and running gear. These halves are permanently coupled back to back by a drawbar and equalizing buffer connection. Each half has its own cab, and carries above the frame one series-wound electric motor, having inter-poles and a divided main-field winding. The large space available for the motor enabled its design to be liberal in all parts, and its location makes the entire motor accessible for inspection. The motor shaft, or axle, carries quartered cranks which are connected by rods to a cranked jack-shaft set between the frames and having its center in the plane of the driving axle center. From this shaft, rod connections are made to the wheels, as in steam locomotives. All moving masses of the rods and cranks are revolving, and are susceptible of accurate counter-balancing. The system adopted for motor control enables one motor to be out of service and the locomotive to be operated in emergency by the remaining motor; also, two or more locomotives may be coupled together and operated as a single unit. The division of the fields into sections for manipulating the field strength, gives four, instead of the usual two, running positions of the speed controller, and thus, also, economizes current during acceleration.

Special attention has been given to the arrangement of control and other apparatus in the locomotive cabs to enable all parts to be readily accessible for inspection and adjustment, such as electrically-driven air pumps for brakes and control, pneumatic sanding devices, contact shoes for the third-rail, overhead pantagraph shoes, sleet-scraping devices for removing ice and snow from the third-rail, automatic train stops for applying the brakes and shutting off power in case of over-running signals in the tunnels, etc. The locomotives are also equipped with boilers for steam generation by electrically heating the water in the boiler to supply the steam-heating system of the trains.

In fixing the capacity of the locomotive, the probable maximum and average train weight was established, and the unit was designed for the most economical distribution of equipment for the Terminal service. It is obvious that on a short run the condition of starting a train from rest and accelerating on the tunnel grades fixes the maximum train which can be hauled, rather than the limitation of motor capacity due to the heating. The maximum weight of train to be hauled by one locomotive under the given conditions was specified as 550 tons trailing

load; the actual capacity, however, in intermittent service, has approximated 700 tons trailing.

A sample locomotive of this design was built, placed under road test in October, 1909, and run 15 000 miles on a continuous test, with a train of 400 tons trailing; also, complete dynamometer-car tests were made of the hauling capacity, speed, and motor characteristics. The detailed design of the mechanical portion of these locomotives was made by the Motive Power Department at Altoona, and the running gear and cabs were built complete at the Juniata shops.

In speaking of this development, it is not out of place to refer to the part taken by the Westinghouse Electric and Manufacturing Company, the contractor for the electric apparatus of the locomotives. From the first this company not only co-operated with the Committee in all needful respects in furnishing suggestions and information, but built at its own expense the electrical portions of the first two electric locomotives, and subsequently a complete locomotive of another type, all of which it placed at the disposal of the Committee for service and other tests.

The principal dimensions, weights, etc., of the adopted type of locomotive are:

Weight per driving axle..................	48 750 lb.
Total weight on drivers..................	195 000 "
Weight on each 4-wheel truck............	57 500 "
Total weight of complete locomotive......	310 000 "
Total length over all.....................	64 ft. 11 in.
Rigid wheel-base of each half............	7 " 2 "
Total wheel-base of each half............	23 " 1 "
Total wheel-base of locomotive...........	55 " 11 "
Diameter of drivers......................	72 "
Diameter of truck wheels................	36 "
Height from track to top of cab..........	13 " 1 "
Width of cab..........................	10 " 6 "

The weights do not include the electric steam generators for train heating. The tractive effort per locomotive = 60 000 lb. for ½ min., and 50 000 lb. for 2 min., or 12 000 lb. at 800 amperes, all with full field. One of the conditions was that the locomotive was to start and accelerate a 550-ton train, in addition to the locomotive, on the 1.93% maximum tunnel grade, and, with a 550-ton train on level tangent track was to attain a speed of 60 miles per hour. Each locomotive was to have

two direct-current, field-controlled, inter-pole, series motors, with cast-steel frames.

Weight of each motor complete with cranks. 43 000 lb.
Height of center of gravity from track......8 ft. $2\frac{13}{16}$ in.
Top of motor frame above cab floor.........5 " $6\frac{1}{2}$ "

Table 15 is a comparison of the centers of gravity and the weights of electric and steam locomotives.

TABLE 15.—COMPARISON OF ELECTRIC AND STEAM LOCOMOTIVES.

	Experimental, double-truck electric	Adopted articulated electric.	P. R. R. "Atlantic" type, steam.	P. R. R. "American" type, steam.
Total weight (excluding tender), in pounds.	195 140	310 000	176 600	138 000
Height of center of gravity of complete locomotive, from rail, in inches..............	42.5	63.75	73	63
Height of center of gravity of running gear, from rail, in inches........................	*28.0	30.2	33	29
Percentage of weight of running gear below springs to total weight....................	*50.0	16.7	22.7	22.7

*Includes weight of motors.

TELEPHONES AND TELEGRAPH.

Telephones.

The telephone system comprises a complete installation for official and public uses, and was planned jointly by the Telegraph Department of the Pennsylvania Railroad Company and the New York Telephone Company.

Official Equipment.—The official equipment provides for service between the various Departments of the Terminal Division and of the Long Island Railroad. It consists of a main private branch exchange switch-board having trunk lines to the Telephone Company's general exchange system; tie lines to other private branch exchanges in the various offices of the Railroad Company in New York and other cities; extension lines to the various offices of the Terminal and Long Island Railroads in the Station Building; and a special switch-board for the Information Bureau of the Station, by which calls from the public for information as to arrival and departure of trains are received and answered with the minimum of delay.

The main private branch exchange switch-board in the Station has a capacity of 1 000 lines. It is equipped initially for 520 lines, the following being connected at the present time:

40 Trunk lines to Telephone Company's central exchange;
80 Tie lines to the Railroad Company's offices in New York and Philadelphia;
80 Magneto extension lines to the Signal Cabins and Yard Buildings;
260 Battery extension lines for offices in the Station Building.

The board is arranged for 6 operators, with all connections in multiple; it connects with 25 monitor boards, varying in capacity from 10 to 30 lines, and located in the Power-House, the Sub-stations and the Signal Cabins. In signal cabins where there is more than one board they are in multiple, so that one or all of them can be used to respond to the calls. The Train Dispatcher, having a selector bell-ringing circuit, may call up any one of the interlocking cabins without calling the others, or he may call them all in multiple, and talk to all at the same time.

In addition to the above, a system of magneto-telephones is provided for direct calls through these switch-boards from outlying points on the Terminal Railroad, such as the signal bridges on the Meadows Section, and from each signal in the tunnels. In all, 140 points on the railroad are thus provided with magneto-sets.

Tunnel Sets.—The tunnel sets have been especially designed to be placed conveniently and compactly on the tunnel walls above the side-benches; they consist of magneto-sets in special moisture-proof cases. A key with each set permits of its connection to any one of four circuits extending through the tunnels; one to the main private branch exchange switch-board, a second to the Power Director's monitor-board, a third to the interlocking cabin controlling movements on the section in question, and a fourth is spare. There are 53 of these sets at an average of about 1 500 ft. apart in the tunnels. In addition to the four circuits above mentioned, there is a fifth circuit in each tunnel to which are connected a number of loud-ringing gongs for signalling employees, who may be called to the nearest telephone, by a code signal.

Pay-Station Service; New York Telephone Company.—The pay-station service is of three kinds. First, the standard service, where a patron makes a call from a booth; this is handled through switch-

boards and booths in the main waiting-room, the sub-waiting-rooms, the exit, and the Long Island 33d Street concourses. Second, a restaurant pay service, where a patron makes a call from a table while dining. Third, train pay-station service, by which a call may be made from a telephone in a train standing at a station platform. For this latter purpose there are receptacles at convenient points under the platforms, and flexible cable is used for connecting to receptacles on the car.

Installation.—Leaving the main switch-board in the Station, four cables, a 30-, a 50-, and two 15-pair, copper-wire, paper-insulated, lead-covered cables, are run in the yard and tunnel conduit systems to the Hackensack Portal. From a terminal pole at this point 34 open copper wires and a 40-pair cable are carried on the concrete pole line over the Meadows Division to Harrison Sub-station. These lines pass under the Hackensack River by submarine cables. From Harrison Sub-station, connections are made to the New York Division lines to Philadelphia, and through "GY" interlocking cabin to Jersey City; also to the cabins in Manhattan Transfer yard. All open wires are No. 8 and No. 9, B. & S. gauge, copper, and are transposed on the poles at frequent intervals. The cable lines are No. 13 and No. 16, B. & S. gauge, copper wire.

Eastward from the Station, a 15-pair cable runs in each of the four tubes, and in one tube (No. 4) a 110-pair cable runs to the Long Island City shaft, at which point it divides to run to the main power-house and to Sunnyside Yard. In Sunnyside Yard a complete service is provided to the various yard buildings and interlocking cabins. The New York Telephone Company's cable connections are made with the Chelsea Exchange, through cables brought out at a telephone manhole in Eighth Avenue, near 31st Street, and, in addition, two ducts in the East River Tunnels have been leased by the Telephone Company for service on Long Island.

Ringing current for the switch-boards throughout is supplied by the Telephone Company from their City Exchange, but the telephone-operating current is obtained from storage batteries charged by motor-generator sets in the Station, converting 440-volt alternating to 24-volt direct current.

The cables, the pole-line wiring, and the tunnel sets were installed by the Chief Engineer's Department, and the switch-boards and instruments by the Telephone Company.

Telegraph.

The telegraph system was planned by the Telegraph Department of the Pennsylvania Railroad; it connects with the general system of that Road and of the Long Island Railroad, and includes, for the convenience of the public, Postal Telegraph Cable Company's connections and offices in the Station.

A main telegraph office on the second floor of the Eighth Avenue side of the Station is equipped with a two-section, twenty-five lines each, double-jack switch-board, with the necessary lamp-receptacle resistance-panels to the batteries. The power-room is back of the board, and comprises duplicate motor-generators, with batteries, for furnishing power to the lines, the train describers, local sounders, and other low-voltage circuits. The board is laid out for three 6-position telegraph tables, of which two are now in service, and the through wires from the various Pennsylvania Railroad and Long Island Railroad offices connect with them. These wires are used for messages, train reports, and general information. A separate board is also provided in the Station Master's office, and consists of one 12-circuit, single jack-board and 4-position table.

In order to reduce the number of telegraph offices in the Station and yard, the pneumatic tube system, elsewhere described, connects with the main telegraph office, to which messages may be sent for transmission. For the use of the heads of the Departments in the Station, a message-call service, consisting of a 50-needle, individual, set-back annunciator, has been installed in the main telegraph office.

It was at first planned to conduct train movement by telephone, but after due consideration, on account of the new operation involved, and the fact that it was desired to select experienced "block" men, who were telegraph operators, from the existing portions of the railway system, it was thought best to provide them with telegraph block wires, by which means the Chief Train Director could hear all orders over the circuits. A relief telephone block circuit, however, is provided over the Meadows Section, to be used when the telegraph block wires are busy, or when repairmen wish to communicate with each other between interlocking cabins.

Allied to and supplementing the telegraph system, instruments called "train describers" are used in the interlocking cabins from Newark, N. J., to Winfield, Long Island. They constitute a mechanical

telegraph system, registering by disks on dials different train designations and routes. Each instrument has sixteen disks, for as many designations, and one set of instruments is used for in-bound and another for out-bound movements.

Telautograph System.—For the purpose of transmitting messages or orders in written form as a matter of record, or to supply advance information of train movement simultaneously to various offices and interlocking cabins, the Gray telautograph system, with 3 transmitting and 17 receiving stations, has been installed in the Station. The transmitting instruments are in the Train Dispatcher's office, in the main interlocking cabin ("A"), controlling the west throat of the terminal yard, and in interlocking cabin ("B") controlling Long Island Railroad movements. Instructions as to special train make-ups, changes in destination platforms, etc., are thus given to all parties interested, with minimum loss of time. In order to be certain that the message is received, an answer-back system is provided; this is operated by push buttons and bells in the receiving stations and a needle annunciator at the transmitting station. Electric power to operate the telautograph and call-back systems is furnished from the motor-generators in the main telegraph office.

Signalling.

As the rapid movement and safe control of trains over the Division, and especially in the Station yard, are essential to the regular and safe operation of the railroad at its required capacity, the switch and signal system was made unusually complete. The general requirements were laid down by the "Joint Operating Committee," through a Sub-Committee on Signals appointed by it, for the purpose of giving the question the necessary detailed study. This latter committee consisted of Messrs. A. H. Rudd, Signal Engineer, George D. Fowle, Consulting Signal Engineer, C. S. Krick, Superintendent, with the writer as Chairman. A general contract was entered into with The Union Switch and Signal Company for the manufacture and installation of the system, under the supervision of Mr. W. N. Spangler, Supervisor of Signals, in the Chief Engineer's Department.

Yard Conditions.—The physical conditions surrounding the Station yard are exceptional in the following respects:

(*a*) The yard is practically underground, and the view from trains, and of the tower operators, is obscured by numerous yard structures and building columns;

(*b*) Clearances, both overhead and side, between the trains and structures are very limited, and the space available for the installation of signals and apparatus is restricted;

(*c*) The character and extent of the train movements required the realization of the fullest capacity of all track facilities, necessitating the greatest freedom for simultaneous parallel movements;

(*d*) The complexity of the track plans and the presence of yard service facilities, such as the piping system for steam, air, and water, drainage and traction conductors and conduits, required that the signal appliances should be mounted on special foundations, and that control wiring should be run in permanent and accessible shape in a conduit system.

On account of these conditions, it was found necessary to divide the control of the yard movements between four different power-operated switching cabins; one near the west end of the yard, for movements from the North River Tunnel portals to Eighth Avenue; one near, and east of, Eighth Avenue, north side, for the important tail switching for Long Island Railroad suburban trains; one at the Seventh Avenue portal of the 33d Street Tunnels, where the yard tracks are gathered into groups leading into two of the East River Tunnels; and one similarly located under Seventh Avenue at the throat of the 32d Street Tunnels. These cabins, each controlling only a part of the yard, required a certain amount of interconnection to insure rapid and complete movements in the yard, a result accomplished partly by electric locking between cabins, partly by light indicators on the track models in the cabins, to show how the tracks between them are occupied and when trains are approaching, and partly by central communication by telephone, etc., with the Train Director.

The interlocking machines comprise means for obtaining positive control of all signals by the actual position of the switches which they govern; the automatic control of signals by track conditions in advance of them, and by the position of the next succeeding signal to which the impending train movement leads; the automatic locking of all switches in every route by the entrance of trains on these routes, and the automatic release of switches immediately in the rear of a train passing clear of the fouling points. They, further, include means for giving

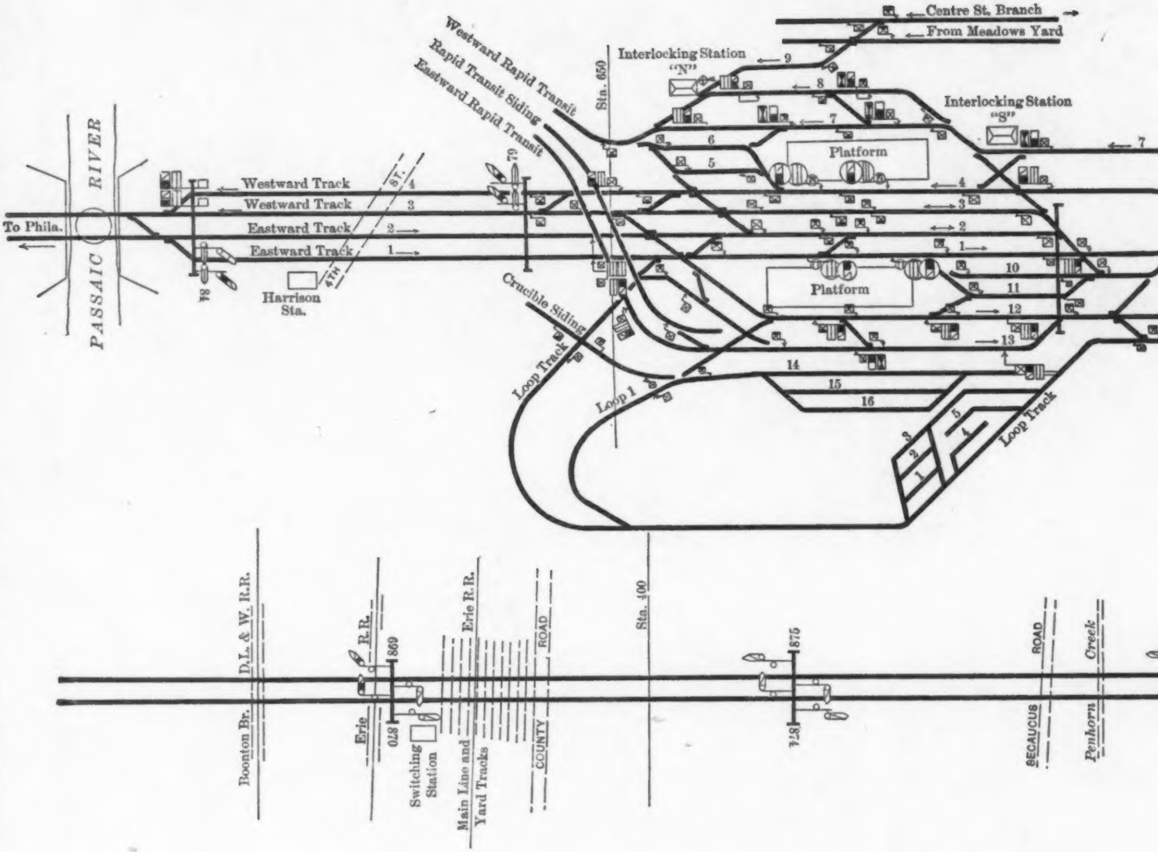
visual indications to the operator of every act of a train in physically locking and releasing the levers controlling switch and signal operations; means for permitting the joint use of all tracks for traffic in either direction between adjacent cabins by co-action of towermen and track circuits; the automatic announcement of trains in their approach to the terminal station through the various tunnels, and the delineation of their movements from block to block through the tunnels, by indicators in each cabin adjacent to the tunnel portals.

Tunnel Conditions.—The tunnels, as elsewhere shown, involve train movements over heavy grades, at high speed, and at the minimum safe interval, a condition which led to the adoption of automatic block signalling with overlaps of the same length as the block sections; in other words, a two-indication block in which a "proceed" indication requires that two sections shall be unoccupied. "Caution" indication must show that at least as much track is unoccupied as the foregoing. The length of the sections is variable, depending on the grade and maximum train speed at the point in question, being made 150% of the length in which a stop can be made by the application of the brakes.

At each block in the tunnels a "track stop" is installed to apply the train brakes automatically should a danger signal be over-run. The closest headway at which trains can be run at normal speed, therefore, is the time required to pass over two block sections, corresponding to about 2 min., and at restricted speed under caution signals, one block, or about 1½ min.

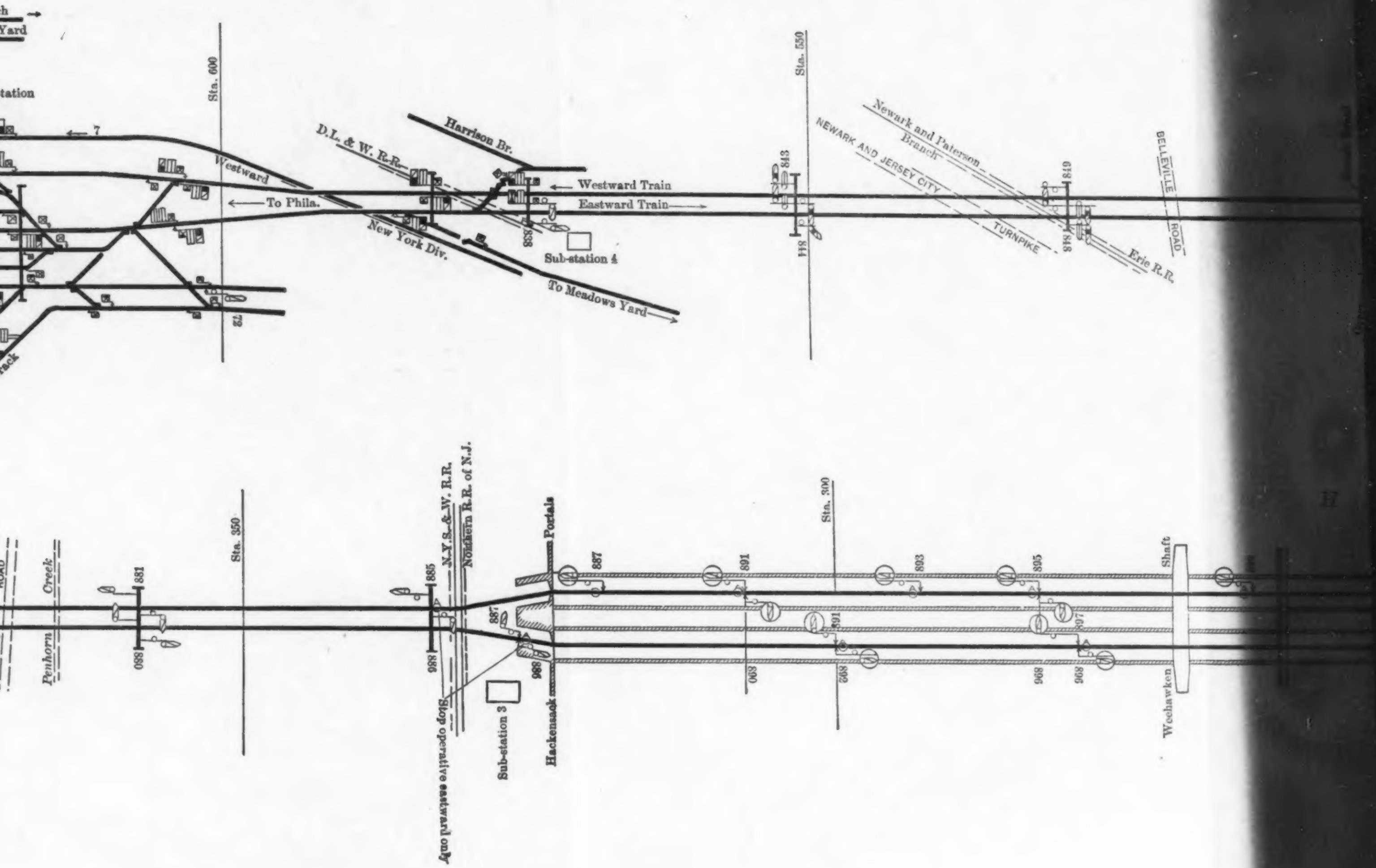
"Lock and block" control has been provided between the Station and the Long Island approach of the East River Tunnels, so that, if necessary, any one of the four tunnel tracks can be operated in the reverse direction. The North River tunnels have the same provision, with the addition of automatic signals for following movements. The grades are such that the spacing of signals for reverse movements could not be the same as for movements in the normal direction of traffic, and, on account of this, there was considerable complication in putting the signals for a certain direction out of commission and those for the opposite direction in, and changing the control of the automatic stops so as to make them effective at the right time.

Meadows Section.—The Meadows Section of the road (Plate CIII) has been equipped with automatic block signals, according to the same principles as those for the tunnels, without track-stops. Complete reverse-movement signalling has been provided for at each block. At



Centre St. Branch
From Meadows Yard
Interlocking Station "N"
Interlocking Station "S"
Westward Rapid Transit
Rapid Transit Siding
Eastward Rapid Transit
Sta. 650
PASSAIC RIVER
To Phila.
Westward Track 4
Westward Track 3
Eastward Track 2
Eastward Track 1
4TH ST.
Harrison Sta.
Platform
Platform
Crucible Siding
Loop Track
Loop 1
Loop Track
Boonton Br. D.L. & W. R.R.
Erie R.R.
Switching Station
Main Line and Yard Tracks Erie R.R.
COUNTY ROAD
Sta. 400
SECAUCUS ROAD
Penhorn Creek

DIAGRAM OF TRACKS AND SIGNALS, MANHATTAN TRANSFER TO TENTH AVENUE.



NALS, MANHATTAN TRANSFER TO TENTH AVENUE.

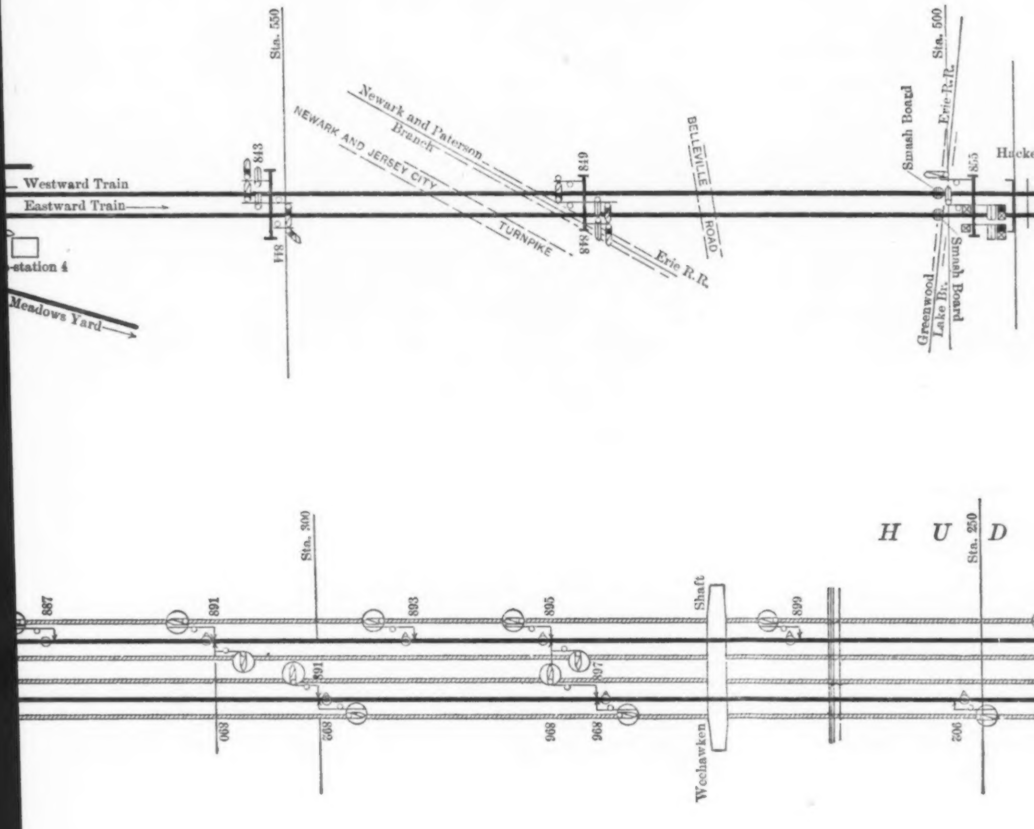
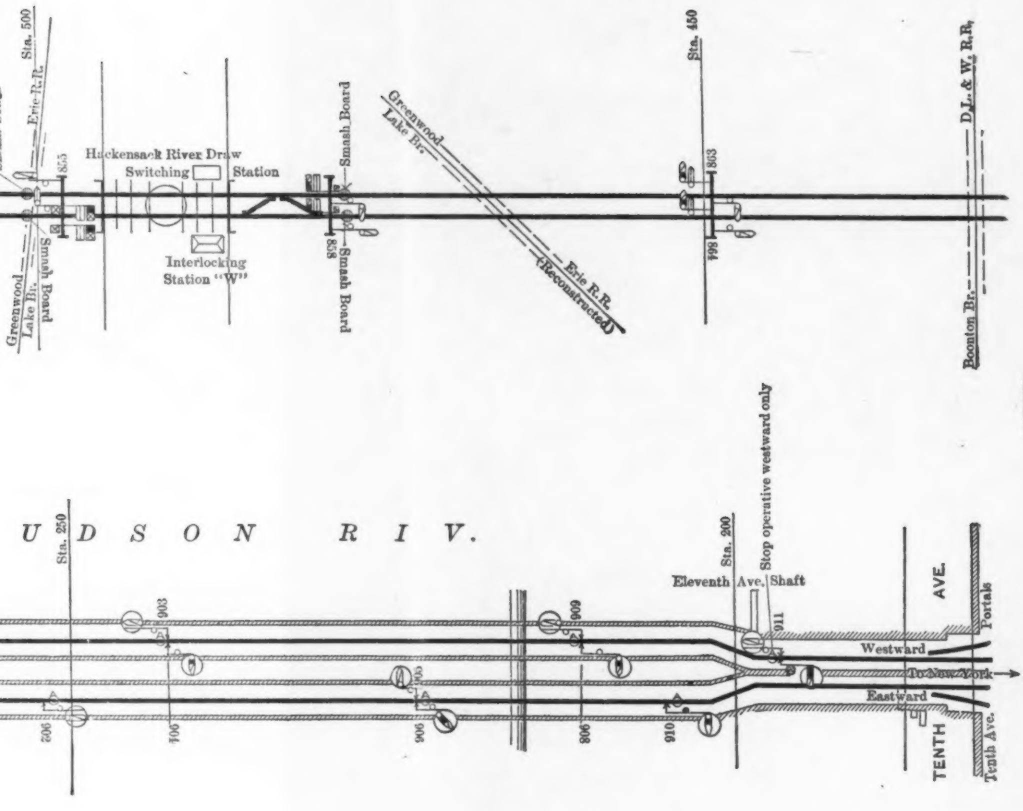


PLATE CIII.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.



the Hackensack River, midway of the section, the drawbridge has been provided with the usual interlocking bridge and signal appliances, and the cross-overs between tracks at the east approach to the bridge, so that reverse movements may originate either east or west from this point.

Outlying Yards.—At Manhattan Transfer, where the interchange of steam and electric power, and the Rapid Transit connection to Jersey City and the down-town district of New York, are made, two very complete electro-pneumatic interlocking plants are provided, controlling all main-line switches from the Passaic River to the separation of the New York and the Terminal Divisions at the east end (Plates CIII and CIV). These interlockings required considerable complication of circuit work to permit of high-speed operation through the yard with sufficient advance information for the control of movements.

At Sunnyside Yard (Plate CV) four interlockings were required, two for main-line movements to and from the tunnels and for the Long Island Railroad movements through the yard, and two for switching in the yard proper.

Type of Signals.—The "Rudd and Rhea" arrangement has been adopted for the signal arms and lights. A signal consists of a 3-position arm to combine the functions of two signals having but two positions each, with the movements of the arm from horizontal to vertical through the upper quadrant. The fact that the signal is automatic is conveyed by pointing the blade for day indication and by a fixed light underneath, staggered in position, for night indication. Interlocking signals are multiple-arm, with square end and vertical lights; the top arm for high speed, the second for limited speed, and the lower one, or a dwarf signal, for low speed. In the Station yard (Plate CIV) one-speed signalling only is used; that is, a high-speed arm permits movement at the highest permissible speed and a lower or "calling-on" arm is provided for use only when a "proceed" indication cannot be given by the normal (or upper arm), or for movements to the storage yard or stub-end tracks.

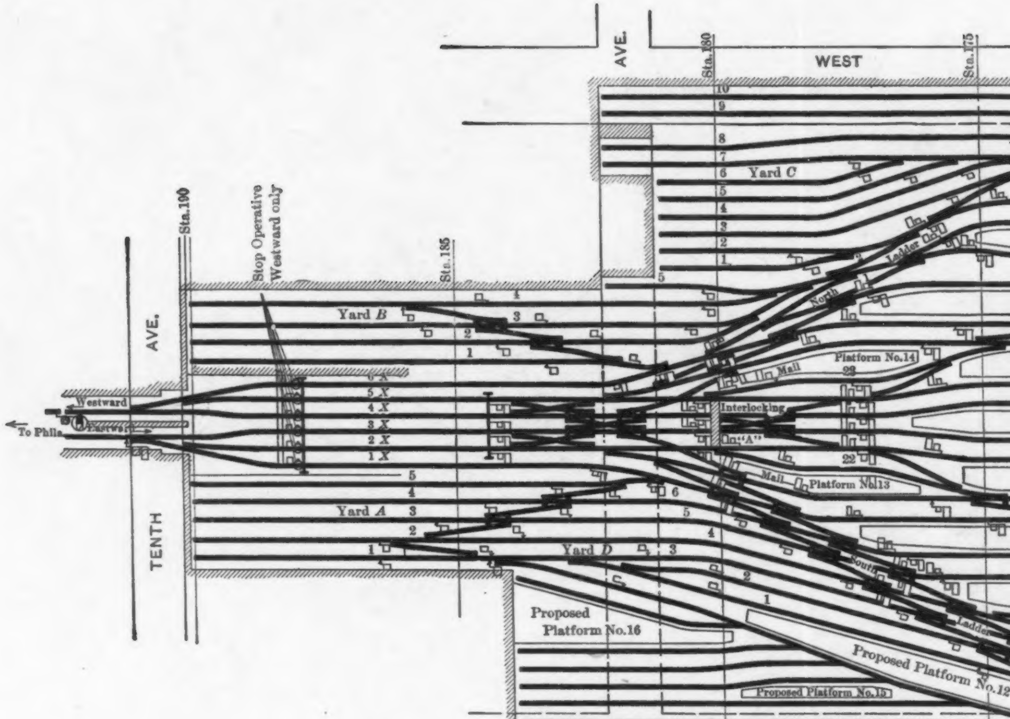
Because of the absence of daylight conditions in the tunnels and in the covered portion of the Terminal area, the use of signals having arms for defining their positions became of secondary importance, and this fact, coupled with close clearances encountered, prompted the total elimination of the arms from all block signals in the tunnels, and from all but the ground or dwarf signals in the Station area, the indi-

cations being given entirely by lights. This signal contains no mechanism other than that required for changing the colors of stationary lights in such a manner as to reproduce the same colors and combinations as called for by changes in position of a semaphore signal under like conditions in the controlling currents. The signals are cast-iron receptacles carrying colored lenses, behind which are located standard 4-c.p. incandescent lamps (two in multiple for each lens), and the mechanism consists of relays, housed in separate shelters near the signals, the contacts of which are adapted to shift the current from lamp to lamp, and thus change the colors displayed as the relays are energized or de-energized by manipulation of the machine levers, or by the action of trains on track circuits, or by both. These relays operate by A.C current in the signal circuit of the track rails, and are in some cases of the pneumatic pin type actuated by A.C. motors, where connected to the track-stops, and in others of the straight A.C. type where signals alone are controlled; both types have multiple contacts, numbering from two to fifteen, as occasion requires.

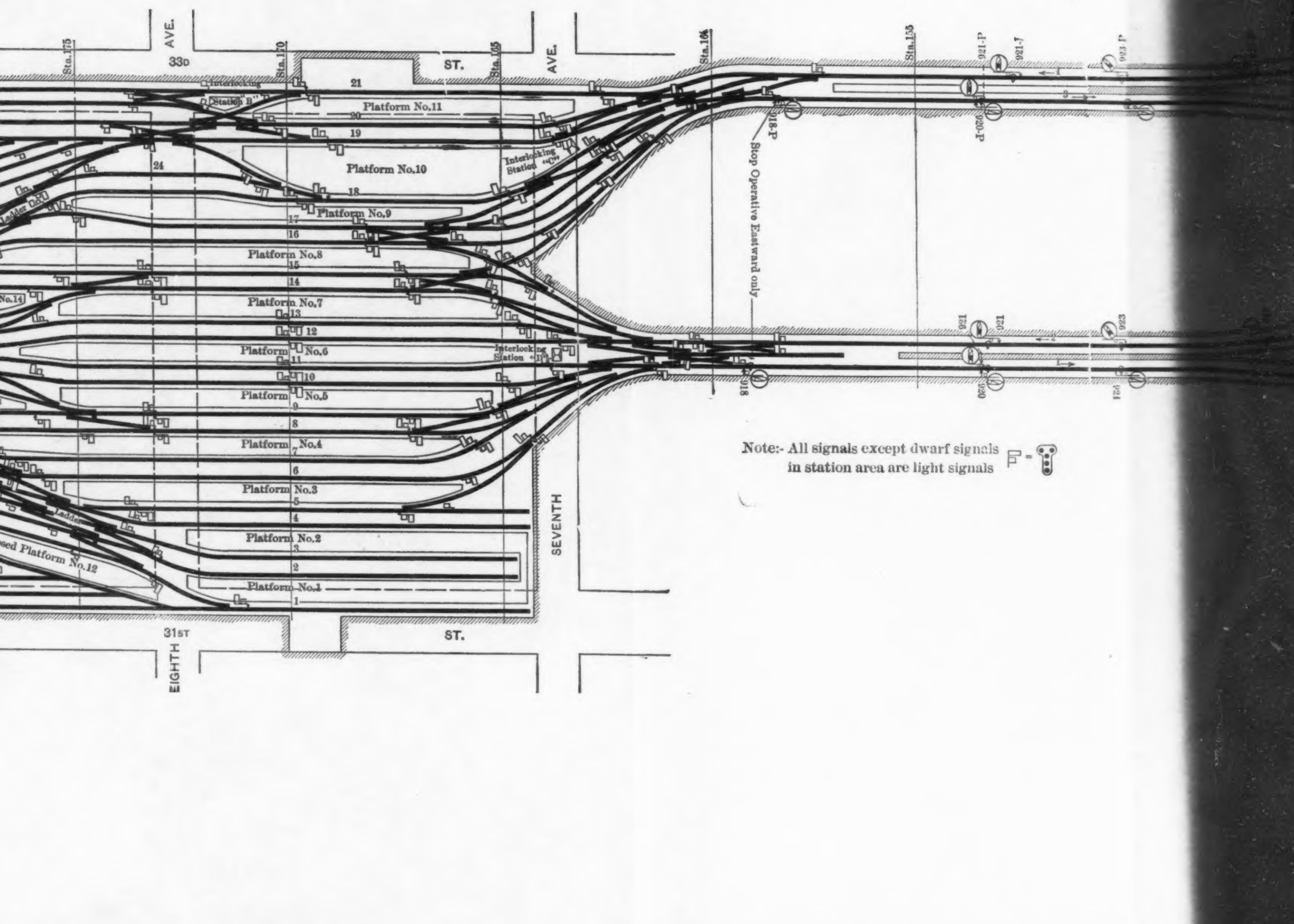
Because of the difficulty of obtaining ample clearances and suitable supports for semaphore signals in the yard, and in order to maintain a uniform type of signal within the Station area, special hooded lenses and lamps of high candle-power are used in the "lamp" type of signal in these exposed places, and are found to give effective indication for the possible range of observance. Elsewhere on the line, outside of the terminal and tunnels, this form of signal would not prove satisfactory because of the higher speeds and longer range of observation required; therefore, the semaphore type is used on the open line.

Circuit Control.—The control of all signals, block or interlocking, is secured automatically by track circuits. In the Station yard various functions were required of these circuits to permit of the most flexible train movements under the obscure physical conditions. They may be classified as follows:

(*a*) Circuits used as a substitute for detector bars;
(*b*) Circuits for switch and signal control;
(*c*) Circuits for route locking, where trains passing home signals electrically lock, in advance, the levers of all switches over which they are to pass, and release the levers for subsequent movements when the point of clearance to the switches has been passed;
(*d*) Circuits for indication on the track models in the cabins.



AVE.
WEST
Sta.180
Sta.175
Sta.190
Sta.185
Stop Operative
Westward only
AVE.
TENTH
Yard B
Yard C
Yard A
Yard D
Westward
Eastward
To Phila.
North
Ladder
Mail
Platform No.14
Interlocking
"A"
Mail
Platform No.13
South
Ladder
Proposed
Platform No.16
Proposed Platform No.12
Proposed Platform No.15
WEST
NINTH



33D
ST.
AVE.
Sta.175
Sta.170
Sta.165
Sta.164
Sta.155
Interlocking Station "B"
Interlocking Station "C"
Interlocking Station "D"
Platform No.11
Platform No.10
Platform No.9
Platform No.8
Platform No.7
Platform No.6
Platform No.5
Platform No.4
Platform No.3
Platform No.2
Platform No.1
Platform No.12
Ladder
Stop Operative Eastward only
918-P
918
920-P
921-P
921-7
923-P
921
923
920
924
SEVENTH
EIGHTH
31ST
ST.
Note:- All signals except dwarf signals in station area are light signals

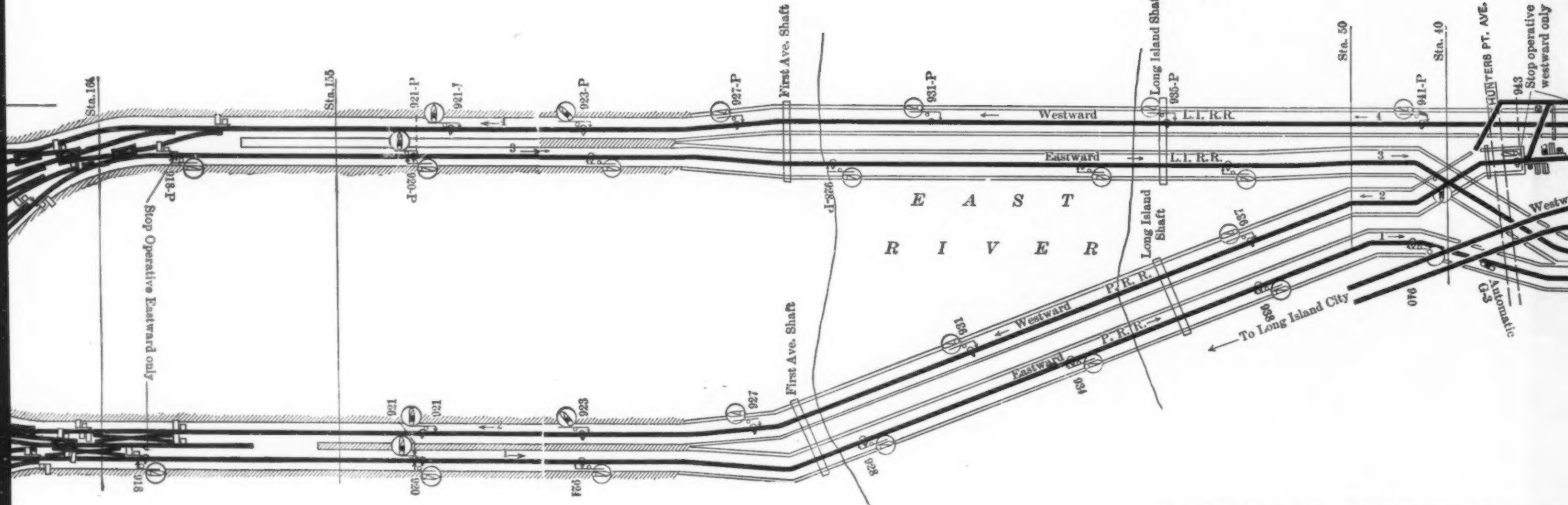
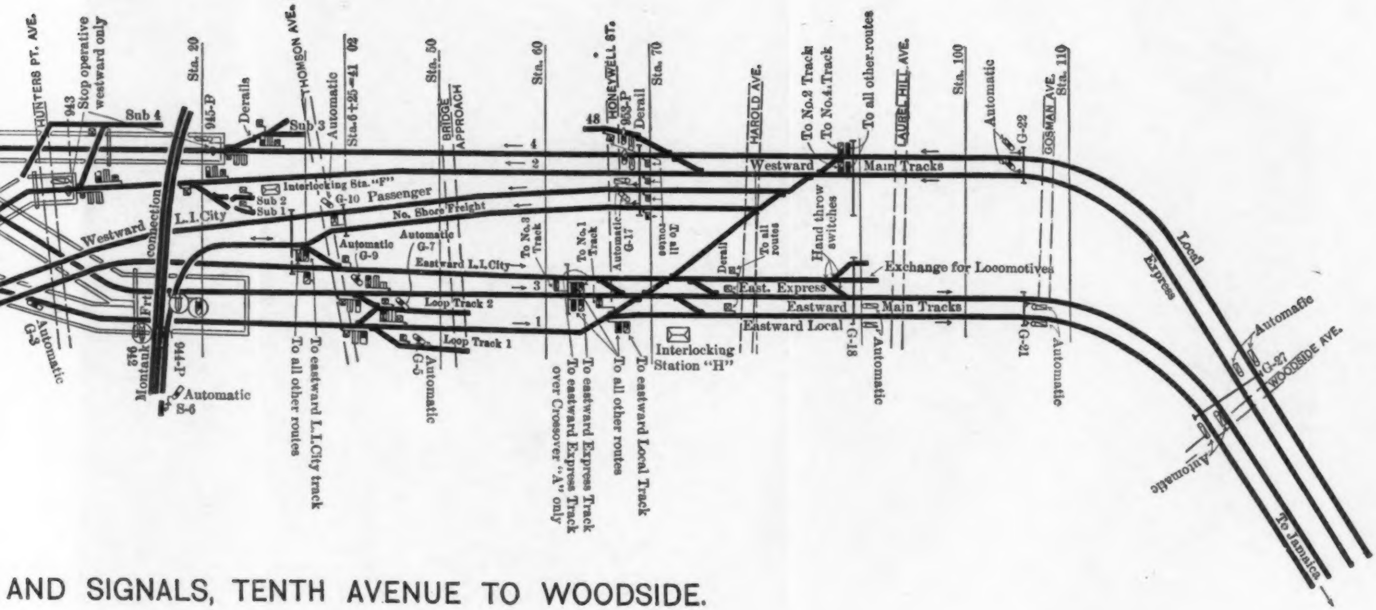


DIAGRAM OF TRACKS AND SIG

Note:- All signals except dwarf signals in station area are light signals

PLATE CIV.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.



AND SIGNALS, TENTH AVENUE TO WOODSIDE.

In addition to the above, secondary circuits are provided for signal control by lever position, by advance signals, by adjacent towers, and by the Train Director through the calling-on arms.

The track circuits are maintained in the rails used for the return current of the traction system, and must, therefore, be of a selective character, operative under all conditions, independent of the presence of the propulsion current. This latter is of the "direct" type, and therefore the signal-control current was made of the "alternating" type, acting upon relays which respond only to this particular kind of current. Continuity of the rail circuit for the propulsion current is maintained at the end of the switch-control sections through copper induction bonds, which allow the heavy propulsion current to pass without material hindrance, and choke back the alternating current sufficiently to maintain the required difference of A. C. potential to operate the signal relays. In the three yards, one rail only is used for the return propulsion current, the other being given up for the signal-control circuits; this is practicable and economical because of the very high carrying capacity of the numerous rails of the parallel tracks, but elsewhere on the main line and in the tunnels, all rails are used for the propulsion return and for the signal operations as well.

Interlocking Machines and Instruments.—All interlocking machines are of the well-known electro-pneumatic type. The control of these machines through the various track and other circuits above mentioned, increases the number of relays much beyond those required by the usual simple interlocking plant, and also requires generally a number of contacts on each relay. Thus, relays are required for: approach locking, and are of the A. C. yard type; an A. C. of the same type of relay for the single-rail track circuits in connection with the locking between cabins and the train-starting system; a D. C. relay, with multiple contacts, for selective and route locking in the cabin machines; a D. C. relay for the light signals; and a D. C. relay for control of the signals through the actual position of the switch points.

A small cabinet is placed over each interlocking machine which, by the presence of a light back of a number, the same as the number of the switch lever, or a number and letter for signal levers, shows when switch levers are unlocked and when signals can be cleared. An electrically-operated track model is also provided. This is a miniature reproduction of the tracks, switches, and signals controlled by the

plant, the switches being automatically moved as the routes are set up, and the occupancy of successive block sections through the tunnels being shown by small lights on the sections of the model. Train describers, consisting of a case with a dial and push button, for announcing departure of trains between cabins; telautographs to transmit special instructions of the Train Dispatcher to towermen; telephone, telegraph, and train-starting system instruments, are all provided in the Station yard cabins.

Power.—Compressed air is used for the mechanical operations in the interlocking plants, for the track stops in the tunnels, and for the signals on the Meadows Section. Electric power operates the control circuits for interlockings, track stop control, block signals in the tunnels and yard, and signal lights on the Meadows.

The main air supply is from the 31st Street Service Plant, and is relayed by compressors at Sunnyside Yard and from the New York Division plant at the Meadows Shops.

The piping system consists of duplicate parallel lines in the Station yard and by a line in each tunnel, with cross-connections for emergency. The pipes are run in the subways under the yard tracks, and at the side of the tunnel benches, and are accessible throughout.

The electric power is from the general auxiliary power system, elsewhere described, the generators, as shown, being relayed at two different power plants. The current is single-phase, 60-cycle, alternating, distributed at 2 200 volts, from a special switch-board in the Service Plant. All mains are in duplicate, and are independent of those used for other purposes. In the yard and tunnels the mains are of rubber-insulated, lead-covered cable, run in a conduit system; on the Meadows and in outlying yards, they are of bare copper, placed on the high-tension power pole line. At the signal cabins and automatic signals, transformers are provided for stepping down the main current from 2 200 to 220 volts for the local systems. This 220-volt current is further stepped down, by special transformers with double secondaries, to 13 volts for the track circuits, and to 55 volts for the signal lamps and other apparatus using A. C. current. In each interlocking cabin, a small motor-generator converts the 220-volt, A. C. to 25-volt D. C., to supply the D. C. relays and electro-pneumatic valves. A storage battery, charged by the motor-generators, maintains constancy of the current supply.

DIAGRAM OF TRACKS AND SIGNALS.
SUNNYSIDE YARD.

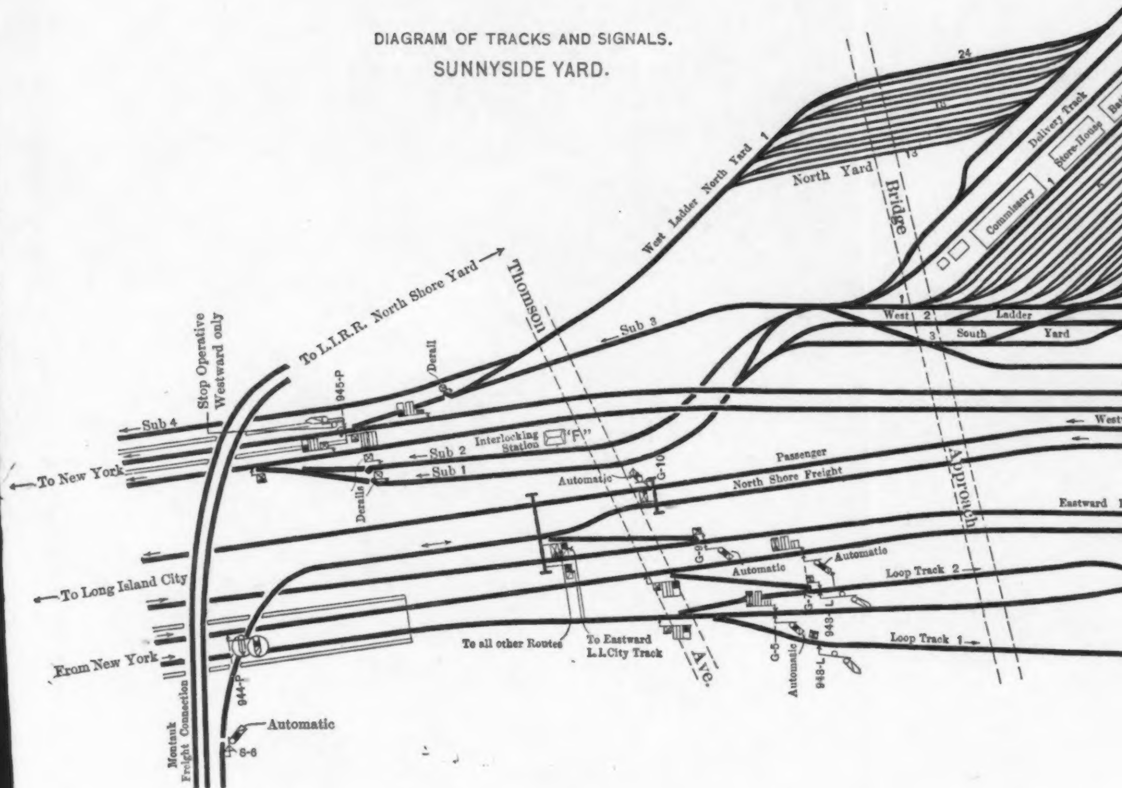
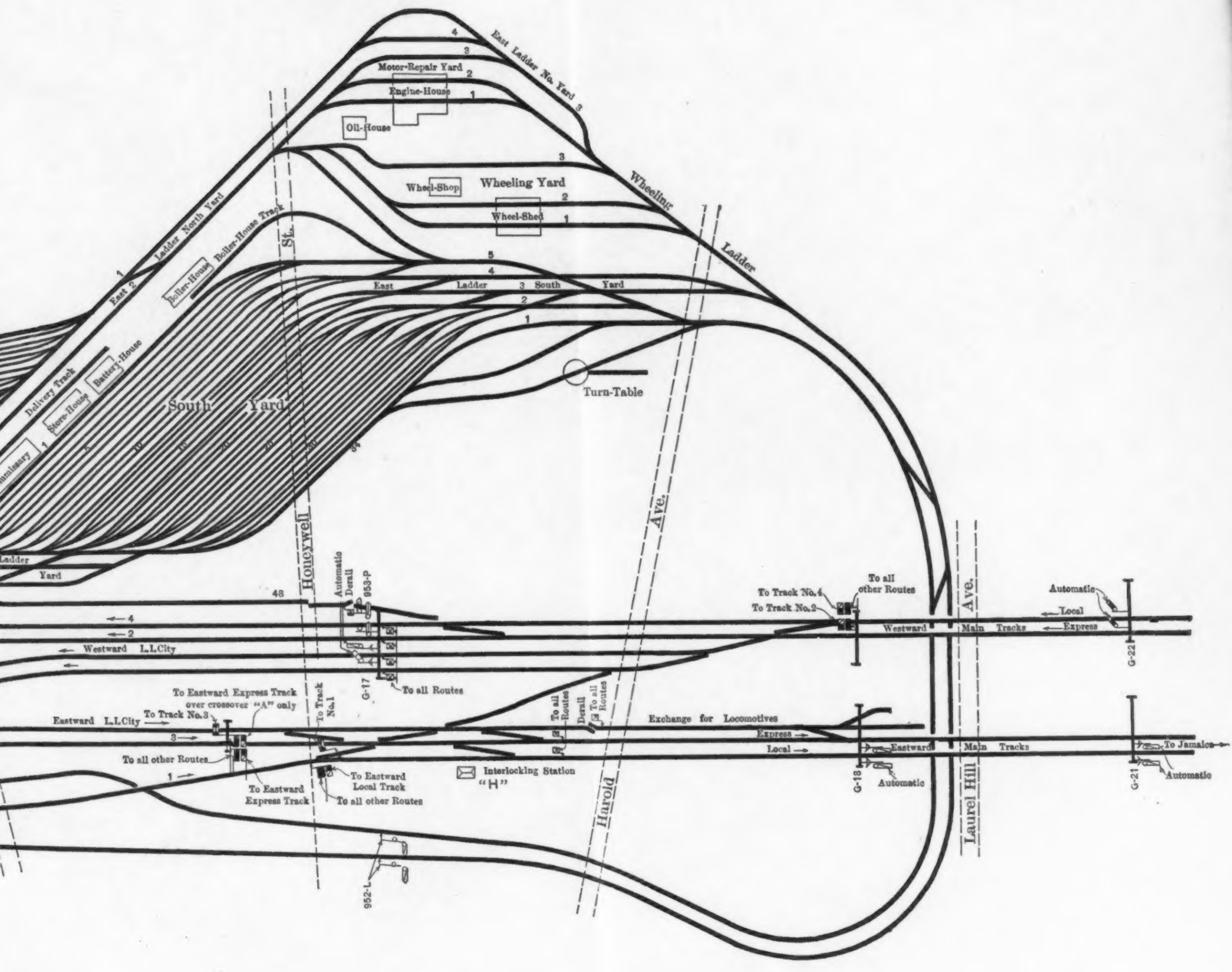


PLATE CV.
PAPERS, AM. SOC. C. E.
MAY, 1911.
GIBBS ON
PENN. R. R. TUNNELS: STATION, TRACK, YARDS, ETC.



Manner of Installation.—Because of the complexity of the Station yard track plan, caused by the presence of traction appliances, service piping, etc., the installation of all switch and signal apparatus and circuit work has been made as permanent and accessible as possible. All groundwork foundations are of concrete, and wires are run in fiber conduits set in concrete. The subways under the tracks, and the high station platforms, provided convenient and accessible space for the air-pipe runs, for a large part of the wiring system, and for housing the instruments in their proper locations, so that, while the quantity of apparatus and wiring in the yard is very great, it is felt that all parts of the system can be reached for ready inspection and repairs.

Table 16 is a summary of the various interlocking plants and the movements controlled.

TABLE 16.—CABIN MOVEMENTS.

Symbol.	Location.	Number of Levers:				
		Switches	Signals.	Traffic.	Spare.	Total.
A	Pennsylvania Station Yard............	68	71	2	38	179
B	" " "	17	13	0	17	47
C	" " "	19	15	2	11	47
D	" " "	23	26	2	20	71
F	Sunnyside Yard......................	11	12	6	18	47
H	" "	14	10	2	21	47
N	Manhattan Transfer..................	28	26	6	11	71
Q	Sunnyside Yard......................	22	18	0	31	71
R	" "	15	15	0	28	59
S	Manhattan Transfer..................	25	23	10	25	83
W	Hackensack River Bridge.............	2	2	5	2	11

The number of relays required for the operation of Cabin "A" is 900, and the total number for all cabins is 2 600. The total length of signal wire used was 1 530 miles.

ORGANIZATION.

The organization for carrying on the work of the writer's Division was developed primarily for designing and later for construction as well.

Committee Work.—The committees of Pennsylvania Railroad operating officers formed the nucleus for the early and general determinations, and, as the work progressed, passed upon general plans and operating methods, submitted to them by the engineering organization. Thus, at all stages of the work, the general officers of the road were in

touch with the Terminal developments, gave valuable advice, and fixed its operating characteristics. The most important Committees were:

First.—The Yard Committee; determining capacity, train movements, etc., of the Terminal, and developing a track plan;

Second.—A Station Committee, formed to consider the building plans as regards the needed operating facilities, sizes of the different rooms, etc.;

Third.—An Operating Committee, determining tunnel size, signal system, and general operating facilities.

These committees had J. T. Richards, M. Am. Soc. C. E., Chief Engineer of Maintenance of Way, as Chairman, and other important operating officers as members. Subsequently, these three committees were merged into a Joint Operating Committee, with Mr. Richards as Chairman.

Fourth.—A Mechanical and Electrical Advisory Committee, with T. N. Ely, M. Am. Soc. C. E., Chief of Motive Power, as Chairman, formed to pass upon the power-system machinery and other matters of a mechanical nature;

Fifth.—A Locomotive Committee, consisting of the General Superintendents of Motive Power, Lines East and West of Pittsburg, the Mechanical Engineer, and the writer as Chairman; to develop a suitable design of electric locomotive;

Sixth.—A Signal Committee, acting as Sub-Committee of the Operating Committee;

Seventh.—Other special committees, such as for Sunnyside Yard, with Mr. F. L. Sheppard, General Superintendent, as Chairman, to report on particular subjects as occasion required.

In order to present to these committees, from time to time, plans and information regarding the work, the writer was made a member of each.

Division Organization.—The actual work of design and construction was carried out by a Divisional organization, under the Chief Engineer of Electric Traction and Station Construction, who was in executive charge of all work described in this paper, and who reported for approval of plans, for authority to incur expenditures, and to carry out the work, to Samuel Rea, M. Am. Soc. C. E., First Vice-President

of the Pennsylvania Railroad. The work of this department consisted of: 1st, designing, both directly and by supervision of the plans of architects and outside engineers employed by contract; 2d, construction at first hand; and 3d, administration of all construction contracts. The manner of executing only the most important separate sections of the entire work can be referred to here in any detail.

The first is the Station Building. The design of this structure was entrusted to Messrs. McKim, Mead and White, Architects, who co-operated, through the Chief Engineer, with Westinghouse, Church, Kerr and Company, the engineers selected to design the steel framework. The building construction was by the George A. Fuller Company, on a percentage contract, executed through a special field organization of the Architects, under Mr. Daniel T. Webster, an associate member of the firm, acting as General Superintendent. This field organization was sub-divided into Superintending and Inspecting, Time and Material Checking, Auditing, and Contract Divisions. The General Superintendent reported to the Chief Engineer for authority to incur each separate expenditure under the main and sub-contracts of the Fuller Company.

The second section is that of miscellaneous outside engineering and construction. This was entrusted to Westinghouse, Church, Kerr and Company, under percentage contracts, allowing such latitude as seemed fit to the Railroad Company in selecting the best manner of prosecuting the work. Under this arrangement the above organization supplemented the staff of the Chief Engineer in much important engineering work, furnishing the designs for the street bridging and viaducts, the sub-structures in the Station Yard, the machinery for the interior services of the Station Building, and the Main and Service Power-houses, as well as other miscellaneous engineering advice. They, further, executed certain construction work, as from time to time authorized, notably, the power-houses and the interior services of the Station. This construction was administered in a similar manner to that of the Station Building, through a field office under George B. Caldwell, M. Am. Soc. C. E., General Superintendent of Westinghouse, Church, Kerr and Company, and reporting to the writer.

The third section covers the designing and construction work done directly by the Chief Engineer's department. This, in general, in-

cluded the determination of plans and reports to the Management, the design and construction of the traction and auxiliary power systems, the installation of machinery in the sub-stations, the Sunnyside Yard and tunnel facilities, and of twenty-eight buildings at various points on the railway; also the laying and ballasting, through the Division Superintendent, of all tracks (except west of the Hackensack Portal), the erection, by contract, of thirty-six buildings, and supervision of the installation of the signal system.

The departmental construction was under the direct charge of Mr. Hugh Pattison, Superintendent of Construction, and his Assistant, Mr. C. G. Edwards, and included the simultaneous prosecution of a great variety of work in the shortest possible time, and in many instances under exceptional difficulties.

The electrical designing work was in charge of Mr. S. A. Spalding, Electrical Engineer; the buildings and structures, Mr. R. D. Coombs, Structural Engineer; and the signal system, Mr. W. N. Spangler, Supervisor of Signals. The inspection of materials and machinery in process of manufacture, and of work under construction, was entrusted to Mr. L. S. Boggs. The trackwork was installed by Mr. C. S. Krick, Superintendent, with his divisional organization under Mr. C. I. Leiper, Division Engineer, and Mr. T. J. Skillman, Supervisor. The general accounting was in charge of Mr. E. J. Bell, Chief Clerk in the Chief Engineer's office. E. R. Hill, M. Am. Soc. C. E., Assistant to the Chief Engineer, was specially charged with the details of the engineering, construction, and inspection work as a whole.

Number of Men Employed.—The following data will perhaps be of interest, in connection with this extensive work:

Average office force employed in designing and supervising work by the Architects', Engineers' and Chief Engineer's offices	272
Average force for inspection materials	40
Average field force (workmen) for last two years:	
Chief Engineer's organization	1 760
George A. Fuller Company	1 800
Westinghouse, Church, Kerr, and Company	1 100
Other contractors	2 980
Maximum number of men engaged simultaneously on work of this Division	8 529
Average number for the last two years	7 641

Contracts.

The Terminal Division construction, herein described, involved the execution of more than 100 formal contracts for the important separate sections of the work, aside from upwards of 100 000 orders, given in the usual manner, for miscellaneous materials. The contracts were made on forms approved by the Legal Department of the Road. The following contractors supplied important machinery and furnishings:

Arc lights	Adams-Bagnall Company.
Ammonia compressors	York Manufacturing Company.
Air compressors	Ingersoll-Rand Company.
" "	Nordberg Manufacturing Company.
Boilers	Babcock and Wilcox Company.
Brick (courts and driveways)	Harbison-Walker Refractories Company.
" (common and enamel)	Sayre and Fisher.
Centrifugal pumps	Jeanesville Iron Works Company.
" "	Henry R. Worthington.
Cabinet work	Brunswick, Balke, Collender Company.
Cabinet work and interior trim	Sloane and Moller.
Cranes	Northern Engineering Works.
Copper cable (bare)	American Steel and Wire Company.
Column casings (Station Building)	J. B. and J. M. Cornell Company.
Clocks	Self-Winding Clock Company.
Cement	"Atlas," "Alpha," and "Giant."
Electric machinery	Westinghouse Electric and Manufacturing Company.
Economizers	Green Fuel Economizer Company.
Electric switches	A. and J. M. Anderson Manufacturing Company.
Elevators (Station Building)	Otis Elevator Company.
" (Sunnyside Yard)	Albro-Clem Elevator Company.
Electric lights	Nernst Lamp Company.
Escalator	Otis Elevator Company.
Excavation (Terminal Yard, and erection of viaducts)	New York Contracting Company.
Electric light fixtures	Edward F. Caldwell and Company.
Frogs and switches	Pennsylvania Steel Company.
Fire-alarm apparatus	Gamewell Fire Alarm Telegraph Company.
Fire-proofing	National Fireproofing Company.
Garbage destructor	Morse-Bolger Company.

Gate-operating device	Burdett-Rowntree Manufacturing Company.
Granite	Norcross Brothers Company.
Glass	Pittsburg Plate Glass Company.
Inspection: Structural steel	William R. Webster, M. Am. Soc. C. E.
Insulated cable	Standard Underground Cable Company.
" "	General Electric Company.
" "	J. A. Roeblings' Sons Company.
Kalamein wood frames and sash	Sloane and Moller.
" " " " "	Manhattan Fireproof Door Company.
Kitchen equipment	Duparquet, Huot and Moneuse.
Locks and hardware	P. and F. Corbin.
Lifts (baggage and passenger)	Standard Plunger Elevator Company.
Lathing and plastering (Station)	H. W. Miller, Inc.
Marble work	Batterson and Eisle.
Miscellaneous marble	J. H. Shipway and Brother.
" "	Traitel Marble Company.
Mail-handling machinery	Lamson Consolidated Store Service Company.
Ornamental ironwork (Station)	Hecla Iron Works.
" " "	Richie, Browne and Donald.
Oil-handling system, Sunnyside Yard	S. F. Bowser Company, Inc.
Paint (for structural steel)	Toch Brothers.
" " " "	Detroit Graphite Company.
Plumbing fixtures	Sanitas Manufacturing Company.
Pumps (Power-houses)	Epping-Carpenter Company.
" "	The Heisler Company.
" "	Henry R. Worthington.
" (Tunnels)	Union Steam Pump Company.
Paving brick	C. C. Hendrickson.
Pneumatic tubes	Interstate Pneumatic Tube Company.
Pipe covering	Keasbey and Mattison Company.
" "	Johns-Manville Company.
Power-house construction and Station services	Westinghouse, Church, Kerr and Company.
Radiators (Station Building)	American Radiator Company.
Refrigerating machinery	Brunswick Refrigerating Company.
Refrigerator boxes	Lorillard Refrigerator Company.

Roofing (skylights)	National Ventilator Company.
" (Station Building)	J. C. McFarland Company.
Rail (track)	Bethlehem Steel Company.
" "	Cambria Steel Company.
Seats (Station Building)	Brunswick, Balke, Collender Company.
Structural steel (Station Building and viaducts)	American Bridge Company.
Steel poles	McClintic-Marshall Construction Company.
Station Building (General contractors)	George A. Fuller Company.
Signal cables	Kerite Insulated Wire and Cable Company.
Signals	The Union Switch and Signal Company.
Safes	York Safe and Lock Company.
Stack lining	M. W. Kellogg and Company.
Steel lockers and equipment	Merritt and Company.
" " " "	Wayne Iron Works.
" " (ticket cases and equipment)	General Fireproofing Company.
Steel fencing (Sunnyside Yard)	Wayne Iron Works.
Spiral mail chutes	Otis Elevator Company.
Third-rail	Cambria Steel Company.
Telephone cables	The Waterbury Company.
" "	Western Electric Company.
Telautographs	Gray Telautograph Company.
Train indicators	National Indicator Company.
Tower foundations (Hackensack River transmission crossing)	F. M. Stillman and Company.
Turbines	Westinghouse Machine Company.
Umbrella sheds (Sunnyside Yard)	Brann and Stuart.
Ventilating fans (for tunnels)	American Blower Company.
" " (station heating)	B. F. Sturtevant Company.
Vault lights	Tucker and Vinton.
Vaults	York Lock and Safe Company.
Valves	Chapman Valve Company.
"	Nelson Valve Company.
Water filters	Loomis-Manning Company.
Yard Buildings (blower-houses and signal cabins)	John W. Ferguson Company.

Conclusion.—The writer, in concluding this lengthy, but inadequate, description of a portion of this great terminal construction, desires

to express his obligations to very many persons, the officials of the Pennsylvania Railroad Company, his staff, and many others. He feels unable to do this adequately, but the following names should be specially mentioned: The late Mr. A. J. Cassatt, President, to whose extraordinary foresight and grasp of detail is due the Tunnel Extension and Station as an accomplished fact, substantially in detail as he had approved it years before its construction; Mr. Samuel Rea, First Vice-President, the Executive in direct charge of the entire project for which he had labored for so many years, and to whom the writer feels under special obligations for support and confidence in his efforts; A. J. County, Assoc. Am. Soc. C. E., in discharging his varied duties as Assistant to the First Vice-President in connection with the work; the Directors of the Company, and its President, Mr. James McCrea, who continued and financed the great work; and to the other Executives, who take possession of the completed project and assume the responsibility of making it a commercial success in its operating and traffic relations. The writer is also indebted to the members of the Pennsylvania Railroad Committees, for valuable assistance, among whom are:

Mr. J. T. Richards, Chief Engineer, Maintenance of Way.

Mr. T. N. Ely, Chief of Motive Power.

Mr. J. R. Wood, General Passenger Agent.

Mr. C. M. Sheaffer, Superintendent, Passenger Transportation.

Mr. R. M. Patterson, Superintendent, Freight Transportation.

Mr. R. L. O'Donnell, General Superintendent, B. & A. V. Division.

Mr. Ralph Peters, President, Long Island Railroad Company.

Mr. J. A. McCrea, General Superintendent, Long Island Railroad Company.

Mr. L. R. Zollinger, Engineer, Maintenance of Way.

Mr. A. W. Gibbs, General Superintendent, Motive Power.

Mr. D. F. Crawford, General Superintendent, Motive Power, Lines West.

Mr. A. S. Vogt, Mechanical Engineer.

Mr. A. H. Rudd, Signal Engineer.

Mr. J. T. Fisher, Superintendent of Telegraph.

Mr. J. C. Johnson, Superintendent of Telegraph.

Mr. H. C. Booz, Principal Assistant Engineer, Branch Lines.

Mr. F. L. Sheppard, General Superintendent, and his staff, for practical assistance in many operating details.

The Division Superintendent, Mr. C. S. Krick, and staff, for constant and helpful suggestions in the design, and for assistance in construction work.

The Architects, Messrs. McKim, Mead and White, especially Mr. W. Symmes Richardson, of that firm, and Mr. D. T. Webster, for able and cordial co-operation.

Messrs. George A. Fuller Company, Mr. A. M. Ganson, General Superintendent, for efficient Station Building construction.

Westinghouse, Church, Kerr, and Company, and especially Messrs. Francis, Caldwell, and O'Brien, of that Company, for expert advice and assistance in a great variety of engineering and construction work.

Mr. F. H. Shepard, Special Representative of the Westinghouse Electric and Manufacturing Company, for helpful assistance.

The many Contractors, for the complete and prompt carrying out of their obligations.

And, lastly, the staff of the writer, especially Mr. E. R. Hill, Chief Assistant, Mr. Hugh Pattison, Superintendent of Construction, Mr. L. H. Barker, Resident Engineer, Mr. S. A. Spalding, Electrical Engineer, Mr. R. D. Coombs, Structural Engineer, Mr. L. L. Newman, Engineer, and Mr. E. J. Bell, Chief Clerk, in the faithful performance of arduous duties.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

MULE-BACK RECONNAISSANCES.

By William J. Millard, Jun. Am. Soc. C. E.

To be Presented September 20th, 1911.

To engineers who have had experience with the mule, which possesses the intelligence of the horse and all the stubbornness of the burro, the title of this paper may seem somewhat ludicrous, but, with all his capriciousness, that animal has proved himself a capable and useful member of surveying parties in Central America. It may be that the writer has been absent from highly civilized countries too long to keep in touch with current literature, but he has never heard of the mule being used in the manner described in this paper.

Reconnaissance work, as far as the writer knows, has never been reduced to an exact science. On railroad work the old and experienced engineer is usually sent out to make the reconnaissance, and this he does in walking or riding over the country. By his experience he can tell whether or not a line is possible, as his eye is trained to measure cuts and fills, estimate the cost of bridges, and judge as to curvature, etc. In the United States he can usually obtain a map of the country from the Geological Survey, but in Central and South America, and in other countries which may be said to be in the first stages of development, maps are not obtainable, or are very inaccurate.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The railroad or mining engineer who is sent to the Tropics to report on the feasibility of a railroad line or the opening of a mining concession, disembarks from the steamer at a little port, and is there confronted by the wilderness. The company which has sent him out cannot allow him sufficient time to make a geodetic map of the country. He must get his information quickly and return home with his report. In all lines of business and in engineering there is a fixed relation between the degree of accuracy required and the work itself. In the reconnaissance survey the mule fixes that relation.

There should be two engineers in the party; one could do all the work, but two can observe more and make the work move more quickly. The instruments required are two small aneroid barometers, a pocket prismatic compass (or a plain pocket compass which may be read to 5°), a wrist strap holding a cheap watch reading to seconds, geological hammers, notebooks, pencils, etc. The work may be done with one barometer, but if two are carried it may prevent the delay caused by going back to pick up a known elevation if one barometer meets with an accident or has been jumped up a few hundred feet.

Naturally, the barometers carried should be capable of recording the highest elevation which will be observed, and they should be read to the nearest 2 ft. The geological hammers are necessary, as they aid in determining the character of the rocks, and this may be of great value to the railroad engineer. For example, certain rocks, when exposed to the elements, disintegrate easily and cause serious slides during the rainy season. Then, too, soft rock on the banks of rivers is likely to be eroded in the course of ten years, thus making it necessary to re-locate the line. In Central America the writer has seen the results of such mistakes in location, the cost of maintaining such a road being enormous.

Before leaving sea level both barometers should be read every half hour for several days. This can be commenced on the steamer while *en route*. In Central America the barometer reading is normal at 9 A. M. From that time until about 3 P. M. the readings will increase. This is illustrated by Table 1, which shows typical barometric readings in Guatemala City. At 7 A. M. the readings are 20 ft. low; at 9 A. M. they are normal; at noon they read 55 ft. high; and at 3 P. M. they have reached the maximum reading for the day.

At sea level the normal reading of Barometer No. 1 was 660 ft.,

that is, its constant was —660; and the constant of Barometer No. 2 was —310. Applying the necessary corrections, the figures in the columns headed "Barom. No. 1" and "Barom. No. 2" indicate the number of feet to be subtracted from readings taken throughout the day on the trail. Of course, areas of pressure in the atmosphere cause changes of several hundred feet, but by reading the barometers at the end of a day's journey and again before starting the next morning, the change can be detected. The writer's experience has been that when a heavy change occurs it takes 5 or 6 days to reach its maximum and about the same time to return to normal. It is not of very much consequence to catch these changes while the party is moving, as the difference rarely amounts to 30 ft. a day. As the average daily journey is 15 miles, and for much of the time is 25 miles, 1 or 2 ft. per mile would not be noticed. Using the table of corrections, the writer has checked the barometric heights very closely in a day's journey (within 10 ft. of the elevations printed on the time cards of some of the railroads in Central America). When an elevation of 5 000 ft. above sea level is reached, or when the party makes a change in location of 4° or 5° of latitude, it is well to make out a new table of corrections.

TABLE 1.—BAROMETRIC CORRECTIONS.

Time.	Corr.	Barom. No. 1.	Barom. No. 2.	Remarks.
7 A. M.	+ 20	— 640	— 290	Constant on No. 1 at sea level = — 660 ft.
7½	+ 15	— 645	— 295	
8	+ 10	— 650	— 300	
8½	+ 5	— 655	— 305	Ditto on No. 2 = — 310 ft.
9	0	— 660	— 310	
9½	— 5	— 665	— 315	
10	— 15	— 675	— 325	
10½	— 25	— 685	— 335	
11	— 40	— 700	— 350	
11½	— 50	— 710	— 360	
12 N.	— 55	— 715	— 365	
12½ P. M.	— 60	— 720	— 370	
1	— 65	— 725	— 375	
1½	— 70	— 730	— 380	
2	— 75	— 735	— 385	
2½	— 80	— 740	— 390	
3	— 85	— 745	— 395	
3½	— 85	— 745	— 395	
4	— 80	— 740	— 390	
4½	— 75	— 735	— 385	
5	— 70	— 730	— 380	
5½	— 65	— 725	— 375	

In order to obtain some idea of the rate at which a mule travels, it is advisable to ride him over a measured distance of 600 or 1 000 ft. at his natural walking gait. The writer's experience has been that the

average mule travels $\frac{1}{20}$ mile per min. on ground varying from level up to a slope of 10° from the horizontal. On moderate hills he will make $\frac{1}{25}$, or possibly $\frac{3}{80}$, mile per min. On steep slopes he may make only $\frac{1}{40}$ mile per min., and on certain steep trails in Guatemala the writer has had occasion to allow only $\frac{1}{80}$ mile per min. Usually the rates of $\frac{1}{20}$, $\frac{1}{25}$, $\frac{3}{80}$, $\frac{1}{30}$, and $\frac{1}{40}$ mile per min. will suffice for the slopes generally met. The figure for jog trotting would be about $\frac{1}{12}$ mile per min.

It is the duty of the engineer who carries the watch to record the mule's rate, as indicated on the sample page of notes (Table 2). In order to approximate closely at first, it is well to select with the eye a point 100 ft. ahead (a stump, or tree, or clump of grass), and note the number of seconds required by the mule to traverse that distance. A table can be readily made, showing the number of seconds required to travel 100 ft. at the various rates, and can be kept at the back of the notebook. The time should be recorded whenever the rate changes.

The time is also noted when certain prominent topographic features are passed, and when the course is changed, but not closer than $\frac{1}{4}$ min. The notes and explanations on Fig. 1, a sample page of the notebook, illustrate the method of carrying on the work. The notes are recorded in lead pencil by the engineer who rates the mules. On the left page (Table 2) he records the time, rate, azimuth or compass course, and the readings of the barometers. On the right page (Fig. 1) he sketches the country as far as he can see, and also makes note of the geology. Notes regarding the compass and the geology are made by the second engineer. Whenever he calls out a change in the course, the first engineer records it, together with the time. Of course, the notes are very crude and difficult to make out, and therefore they must be put in order and inked in immediately after stopping work each day.

The sample notes show that Engineers Dawson and Millard left the hotel at San Felipe at 7.31 A. M., going at the rate of $\frac{1}{20}$ mile per min. The barometric elevations were compared, and checked within 10 ft. The course taken read 20° on the compass. At 7.33 A. M. the course changed to 10°, and at 7.33$\frac{3}{4}$ the last street of the village was passed. At 7.37 the grade became steep enough to change the rate to $\frac{3}{80}$ mile per min., and at the same time the course was changed to 0°, etc., etc.

The figures in the column headed "Dist." represent the fractions

of miles traveled on each course, and are deduced for the purpose of plotting the notes. The corrected barometric readings are written in red ink (in italics in the sample reproduced).

TABLE 2.—RECONNAISSANCE, SAN FELIPE TO QUEZALTENANGO.

Mule back { Dawson. Millard. Oct. 10, 1909.

Time.	Rate.	Dist.	Azimuth.	Barom. No. 1.	Barom. No. 2.
33¾..........	....	....	270°	3340 *2685*	3000 *2685*
33............	1/20	$\frac{3/4}{20}$	....	....	....
26½..........	1/30	$\frac{6\frac{1}{2}}{30}$	310°	....	....
23............	Started.				
22............	Stopped.				
21............	1/40	$\frac{6\frac{1}{2}}{40}$	315°	....	....
18¼..........	....	$\frac{2\frac{3}{4}}{25}$	300°	....	....
15½..........	....	$\frac{2\frac{3}{4}}{25}$	275°	....	....
10............	....	$\frac{5\frac{1}{2}}{25}$	290°	....	....
6¾..........	....	$\frac{3\frac{1}{4}}{25}$	310°	....	....
4............	....	$\frac{2\frac{3}{4}}{25}$	325°	....	....
2¼..........	....	$\frac{1\frac{3}{4}}{25}$	330°	3375 *2725*	....
8 A. M.......	....	$\frac{2\frac{1}{4}}{25}$	300°	....	....
56............	1/25	$\frac{4}{25}$	....	....	....
53½..........	1/30	$\frac{5}{60}$	320°	....	....
46............	1/20	$\frac{15}{40}$	0°	....	....
41............	....	$\frac{15}{80}$	345°	....	....
37½..........	....	$\frac{21}{160}$	350°	....	....
37............	3/80	$\frac{3}{160}$	0°	....	....
33............	....	$\frac{4}{20}$	10°	....	....
7.31 A. M.....	1/20	$\frac{2}{20}$	20°	3165 *2520*	2815 *2530*

The accuracy of this method of surveying is surprising. The writer has had the opportunity to check his work with geographical maps, and found no discrepancy when plotted on the scale of 1 mile to the inch. On a reconnaissance from Quezaltenango to Chiantla, in Guatemala, and back to Quezaltenango, a circuit of 81 miles was made. For six hours on the first day out it rained so hard that it penetrated the

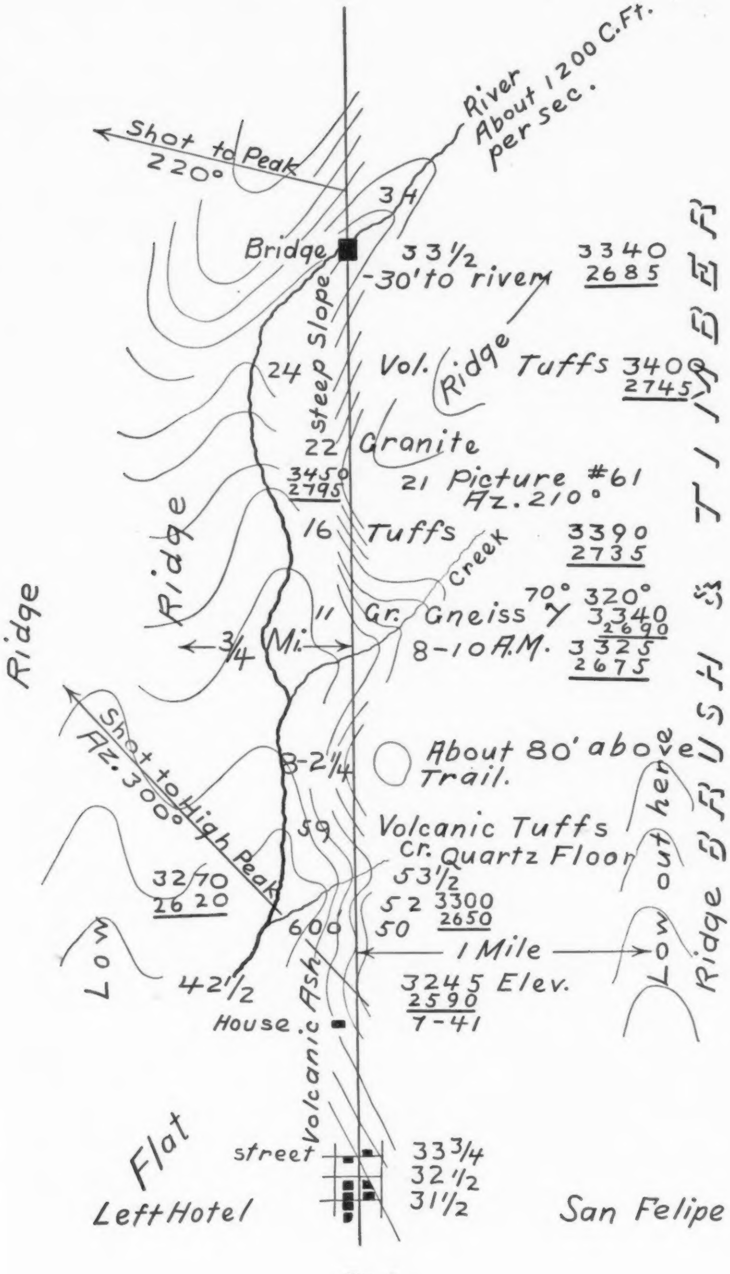
River
About 1200 C.Ft.
per sec.
Shot to Peak
220°
34
Bridge
33 1/2
-30' to river
3340
2685
steep Slope
24
Vol.
Ridge
Tuffs 3400
2745
22 Granite
3450
2795
21 Picture #61
Az. 210°
16 Tuffs
3390
2735
Creek
Ridge
Ridge
70° 320°
11 Gr. Gneiss
3340
2690
3/4 Mi.
8-10 A.M.
3325
2675
Shot to High Peak
Az. 300°
8-2 1/4
About 80' above
Trail.
59
Volcanic Tuffs
Cr. Quartz Floor
53 1/2
3270
2620
52 3300
2650
600
50
Low
1 Mile
Low out here
Ridge
Low
42 1/2
Volcanic Ash
3245 Elev.
2590
7-41
House
Flat
street
33 3/4
32 1/2
31 1/2
Left Hotel
San Felipe

Fig. 1.

oil-skin slickers worn by the party. The trail was very mountainous and the $\frac{1}{80}$-rate was used frequently. The error in closing was only $\frac{3}{8}$ mile, and fell within the limits of the town site. The compass readings were seldom taken closer than to 5 degrees. In addition, the survey was plotted with topography extending to 4 and 5 miles on either side of the trail. Owing to the scarcity of heavy brush and timber in Guatemala the topography could be sketched very well, but, even in Nicaragua, where the brush is very thick, shots may be taken to prominent peaks and ranges of hills, and, when plotted, furnish data much more reliable than the memory.

The crude map thus produced certainly helps the railroad engineer to eliminate routes for preliminary surveys, which might otherwise be undertaken, and the mining engineer can easily reduce to order the mining, prospecting, and other geological information.

The cost of such a reconnaissance is not greater than that of a survey depending on eye and memory only. The party must penetrate the country, and whether it travels by canoe, on foot, or on mule back, the cost of taking notes in the form described herein is nothing. The information obtained, however, may be such as to warrant discontinuing all work, thereby saving money which might have been spent on further surveys of a more substantial character. Under the right conditions, it is essentially the survey for the engineer who has in mind the proper relation between time and money.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

SOME TESTS OF LARGE STEEL COLUMNS.

Discussion.*

By Messrs. Albert Lucius, Leon S. Moisseiff, A. W. Carpenter, J. S. Branne, Lewis D. Rights, J. R. Worcester, Horace E. Horton, A. N. Talbot, James Christie, N. R. McLure, R. S. Chew, and George N. Cole.

Albert Lucius, M. Am. Soc. C. E.—The general facts shown by Mr. Howard's valuable paper are that the columns tested in a satisfactory manner up to the elastic limit of the material, and then one failed suddenly. The tests of the others were not brought to a conclusion. Mr. Lucius.

The details of the action, under the test loading, of the plates and angles composing the column sections, are brought out clearly by the measurements and the accompanying reasoning and explanations, but the reason for the failure, at unit stress approximately representing the elastic limit, is not brought out, and yet it is the most important result of the test that the column did fail at that limit. It is true that the practical value of compression members, as parts of a structure, ceases at the elastic limit of the material, in the same way as the practical value of tension members ceases at this limit, and that, for all purposes of "sectioning," the elastic limit of the material should govern, and not the ultimate strength; but, as far as concerns the designer who is proportioning the several members of a structure, it is probably equally true that it is to him of the utmost importance to develop the design and detail of all compression members the same as of tension members, so that as wholes they not only

* This discussion (of the paper by James E. Howard, Esq., published in *Proceedings* for February, 1911, and presented at the meeting of April 19th, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lucius. act consistently up to the elastic limit of the material in all their detail, but hold out against ultimate failure, as wholes, to the last possible limit. The columns tested by Mr. Howard evidently did not do this.

Bearing in mind the fact that the test columns were duplicates of actual bridge posts to be connected to the trusses through gusset-plates and riveted joints, for which purposes they were probably originally proportioned, and that their change to pin-connected posts was only for the purpose of making these tests, it still appears that, even for this purpose, the pin bearing was insufficient, for it would naturally much exceed the elastic limit of the material, locally, at the pin bearings and in the plates and details of their attachments, by the time the elastic limit was reached for the sections of the post; and less than this limit can hardly have been contemplated in the tests. The proportioning of the pin bearings and attachments corresponds to practice, as represented by most of the specifications, but it is one of the points on which most of the specifications seem to be inconsistent and invite locally bad details and overstraining.

The feature which most distinguishes the make-up of the columns from the usual design is their cellular construction, the two sides, each consisting of plates and angles forming a channel shape, being stiffened and connected by transverse frames at short intervals, which made them competent, evidently, to permit the development of strains up to the elastic limit in the section of the columns, without material deformation, and without calling in any measurable degree on the lacing, which was unusually light. To this light lacing—or in its stead, the absence of a continuous longitudinal diaphragm—the speaker attributes the cause of the early failure of the columns by crippling at or near the elastic limit. As long as the strains were within this limit, and the material maintained its full resilience, the transverse diaphragms were sufficient and the cellular posts acted properly and normally. What there was of lacing was not called upon for action; it was in a neutral condition and practically free of strain, as the tests showed; but, when the elastic limit was reached and the material became more unstable, it was ready for deformation and needed stiffening to hold the two sides of the column in shape. Then the lacing proved insufficient, and the column collapsed sooner than it should have done. As a bridge post, it was not proportioned to bring out its highest possible resistance to overload, even though it acted consistently and well within the elastic limit.

The speaker also questions the propriety of the assumption of fact that steel in compression has the same elastic limit and yield point as steel in tension. It is probably proved sufficiently true that the modulus of elasticity is the same for both plus and minus straining, but the speaker believes the elastic limit and yield point for steel in

compression to be less by an average of 20% than for steel in tension. This has been his experience in making compression tests, which, however, were carried on without the use of fine instruments, and for general information only. The Bureau of Standards and Mr. Howard can readily settle this point definitely for the benefit of the Profession. Mr. Lucius.

While the speaker fully appreciates the value of the information contained in this paper, he would consider this value to lie more in the elucidation of the behavior of material under compression, generally, than in the development of facts bearing on the proper proportioning of columns. In connection with the general proportioning of columns and compression posts, the speaker believes: that the more a column deviates from the general form and properties of a closed circle or square, the less ultimate resistance will it possess; and that the radius of gyration has no meaning if there is not sufficient shear connection between the masses which determine its value. It is this amount of necessary shear connection, when the resiliency of the material becomes unstable, which column tests are hoped to furnish ultimately.

Leon S. Moisseiff, M. Am. Soc. C. E.—This paper treats of the most important problem of the day in statical engineering. The strength and behavior of large steel columns under load is a matter which vitally affects the safety of structures and the economy of their design. The results of tests on five large columns, each having cross-sectional areas of about 90 sq. in., made and reported on by a testing engineer of Mr. Howard's ability in research work, is an unusual occurrence, even in progressive American engineering practice. Mr. Moisseiff.

It is true that the number of members tested was small, and not sufficient to enable engineers to derive rules for practical guidance in the wide field of column application, yet the minute attention paid to the behavior of the columns when tested, and the extensometer observations taken on many individual portions of the member, make these column tests especially valuable; the tests were not quantitative, but qualitative.

The paper is full of dormant suggestions, and contains the elements for a few possible studies. It is to be regretted, therefore, that the stress-strain diagrams of the material of the plates and shapes used in building up the columns (Plates XV to XIX) do not impart the information they are apparently intended to convey. The lines purporting to represent the stress-strain diagrams of the tested specimens do not seem to be drawn to any scale. The speaker has tried to establish the scale to which these curves were plotted, but has failed completely. What purpose do these forty or more curves serve if they do not convey to the reader a graphical picture of the relation of strain to stress? The value of the information presented in the paper is sufficient to warrant drawing the stress-strain curves of the component material of the columns to correct scale.

Mr. Moisseiff. It would also be interesting to know what is meant in these diagrams by the "corrected zero line."

The compressive strain, in stepping down from pin-plate to web, was found by the author to be from 0.001 to 0.002 in. greater than that observed on the web. It developed a perfectly elastic behavior, and disappeared with the removal of the load. This may be rationally explained by the following considerations: The pin-plates, when performing their function and transmitting pressure from the pin to the web of the column, do so by means of the rivets attaching them to the web. The rivets, in transmitting the reaction to the web, are subjected to shearing strains and to bending deflections. The pin-plate itself, in distributing the pressure from the pin to the rivets, is subjected to flexure, and will develop its greatest deflection on the middle line, where the strain was observed. These three strains will combine to cause a greater displacement of the edge of the pin-plate at the middle line than that due to the compressive stresses alone.

In attempting to evaluate numerically the pin-plate strains the following figures are obtained: The total load on the column was 800 000 lb., or one-half on each jaw. The portion transmitted—on the assumption of a uniform distribution—by the outside pin-plate to the web was 82 000 lb. There being 39 rivets provided for this purpose, each rivet carried 2 100 or 3 040 lb. per sq. in. Taking the coefficient of elasticity of the rivets in shear at 12 000 000, the displacement of the pin-plate, relative to the web, due to the shearing distortion of the rivets, becomes 0.00025 in. The displacement due to the flexure of the rivets was only 0.00005. The greatest deflection of the pin-plate, however, due to its action as a distributing beam, becomes 0.00118 for a coefficient of elasticity of 29 000 000 and a depth of 22.75 in. measured on the middle line. The total strain of the pin-plate other than that due to direct compression becomes, therefore:

Due to shearing strain of rivets..........	0.00025 in.
Due to flexure of rivets..................	0.00005 "
Due to flexure of pin-plate...............	0.00118 "
	0.00148 in.

or, say, 0.0015 in. This is the average of the excess in the observed strains.

Evidently, the computed strain is generally sufficient to explain the increase in strain observed in the tests. It is probable, from the indications of the observations made, that, with the metal surrounding the pin reaching higher stresses, the flexure of the pin-plate was considerably enhanced, and that to it was due the observed strain. The effect of the rivet distortion may be neglected as secondary in importance. These figures are used merely as an illustration and only to show relative values.

As the deflection of the plate is in inverse proportion to its moment of inertia, and as the latter varies with the cube of its length, the importance of a long pin-plate becomes apparent. A mere increase of length of pin-plate from its present 27.25 in. to 36 in. would result in an increase of its moment of inertia at the middle line of 2.5 times, and a corresponding decrease in its deflection. The latter would then be reduced to 0.0005 in. Mr. Moisseiff.

Without going into refined computations, the pin-plates in the columns appear to the eye of an experienced detailer to be of insufficient length; nothing less than 36 in. should have been used.

Passing from details, and considering the results of the author's tests at Steubenville, together with those of other tests made in recent years, one arrives at the inevitable conclusion that the ultimate resistance of a well-built steel column is near the elastic limit of its component materials. No pure compression member can reasonably be expected to develop a greater resistance.

By the stress-strain curves of the column, shown in Fig. 6, the author presents bluntly the gradual development of sets characteristic of built compression members which have been exposed to the various methods of manufacturing and shop practice. The question logically arises: Where is the practical elastic limit of such columns? There being no definite point where the laws of proportionality cease, a practical limit, based on past experiences, should be adopted, which would allow, within the "elastic limit" so defined, a certain percentage of the total shortening of the column as a set. Any excess of set would then be considered to be beyond the "elastic limit." The main value of such a conventional ruling would be to furnish a basis of comparison with tension members.

In these days of discussions on lacing bars, it is interesting and instructive to find that the strains observed by the author on the lacing of the tested columns were insignificant. A well-centered, perfectly straight column, subjected to an axial load, needs but little lacing, as long as it remains straight. Inequality in the elastic behavior of the column material is insufficient to require heavy lacing. Assume the extreme condition that the coefficient of elasticity of one channel is 24 000 000, and that of the other is 36 000 000, which values are within those observed by the author on his specimen tests: At a stress of 20 000 lb. per sq. in. of section of the column tested, the tie-plates alone would be able to equalize the unequal resistance of the channels with a shearing stress of 3 500 lb. per sq. in., and, together with the lacing bars, with a stress of about 1 500 lb. per sq. in.

The straightness of a column, however, is a relative term, and here, again, an inquiry is needed as to what would be passed on good inspection as a "straight" column, and how much deviation from a straight line would have to be allowed to meet the contingencies of everyday practice.

Mr. Moisseiff.

The results thus obtained would furnish empiric data, on which a practical column formula could be established, which would allow for possible variations in the coefficient of elasticity, reasonable lack of straightness, shop inaccuracies, and unavoidable deformation in transportation, and, at the same time, would furnish a rational basis for design of column lacing.

Meantime, before such inquiries have been made, engineers will have to use their judgment as to the proper stresses to be allowed on columns, and their reduction for increasing lengths. It may be stated here that, during the last two years, the Department of Bridges, of the City of New York, has been using in its work, for columns in axial compression, three-quarters of the stress allowed on tension members and properly reduced. Thus, where 16 000 lb. per sq. in. was allowed in direct tension, $12\,000 - 30 \frac{l}{r}$ was used on members in compression.

Mr. Carpenter.

A. W. Carpenter, M. Am. Soc. C. E.—These tests and the record thereof, in common with some others recently published, are far in advance, in their scope and completeness, of those on which the knowledge of column strength has heretofore been based. They are not merely a determination of the load required to produce failure in the weakest part, but are a searching study of the strain action of the columns in their various parts and the metal therein, under moderate stresses and under increasing stresses up to failure, and all the results are expressed in definite measurements. They show a wonderful advance in the science of testing, and the Profession is to be congratulated in that Mr. Howard has given his attention to the column problem.

The speaker proposes to discuss this paper mainly from the viewpoint of bridge design.

Up to a very recent date, the design of these columns would undoubtedly have been considered beyond criticism. Under some very modern but widely used specifications, however, some of the details, such as the pin-plates, in respect to their length, and the lacing, would be condemned.

The specifications used in the speaker's practice require at least one full-width pin-plate on each segment to extend to the far edge of the stay-plate, and the others to some distance beyond the near edge. The pin-plates in the columns described extended about 12 in. beyond the near edges of the stay-plates, which were 2 ft. 10½ in. long. The speaker's specifications would not call for such long stay-plates, but would require one set of pin-plates to be 12 in. longer. The shortness of the pin-plates may be more clearly realized, perhaps, when it is understood that they had a length of only seven-tenths of the width of the column segments. Except in the matter of length, they fulfill the requirements of the specifications mentioned. The same specifica-

tions require the lacing to be proportioned for a shear, transverse to the axis of the column, equal to $320\ A\ \frac{r}{V}$, in pounds, in which Mr. Carpenter.

A = the area of the column section, in square inches;
r = the radius of gyration of the column section about an axis perpendicular to the plane of the lacing, in inches; and
V = the distance from the axis of the column to the extreme fiber, in inches.

To resist a shear of 26 000 lb., this formula would require 3 by ½-in. lacing, with 2-rivet connections. The columns in question had 2½ by ⅜-in. lacing, with single-rivet connections.

The weakness of the pin-plates was fully demonstrated by the tests, and it would appear that lengthening the plates would have strengthened the columns. There was no apparent weakness in the lacing. It must be confessed that it is difficult to see how heavier lacing could have reinforced Column No. 5 at the point where failure commenced, that is, how the buckling of the web could have been retarded by heavier lacing.

The low values obtained for the strength of these columns is somewhat surprising, in spite of the deficiencies in the details noted. The speaker would have expected values from 10 to 15% higher. The values are no greater than those which have been obtained from the average of recorded tests on wrought-iron columns of structural dimensions. A closer examination seems to show that the explanation of this is in Mr. Howard's theory that the elastic limit is the element which fixes column strength, and that a column will not be much stronger than the elastic limit or the yield point of the weakest component piece. The minimum elastic limit and yield points recorded for the material of these columns are lower than the ordinary similar limits for wrought iron of the same dimensions. Viewed in this light, there seems to be no reason to expect any greater strength from columns of this grade of steel than from similar columns of wrought iron, although the ultimate strength of the steel in tension may be much greater than that of the wrought iron.

To the speaker Mr. Howard's tests indicate that the usual working stress for bearing on pins is taken too high and should be nearer the base constant of the column formula used.

The speaker believes that the influence of form of metal is considerable in determining the strength of a column. The columns in question had nominally straight webs, more or less injured, necessarily, by straightening, punching, riveting, etc. This form is most easily deflected from a straight line by axial compression or transverse pressure. Take the same metal and corrugate it longitudinally, curve it as in the form of the Phœnix column, or bend it frequently as in

Mr. Carpenter.

Z-bars, and greater resistance to compression might be expected; in other words, it could not yield so readily.

The point of greatest interest and value in these tests undoubtedly is their bearing on the safe working stresses for columns. Assuming that we accept the lowest value for yield point in specimen tests of the material of a column as the value for the ultimate strength of a short column as a whole, what is a safe working value? The curves in Fig. 6 show that the stress-strain proportionality begins to vary from about 15 000 lb. per sq. in. upward. What is the significance of this? Does it mean that repeated stresses above—even slightly above—those representing the point of divergence of the stress-strain curve from the straight line (that is, about 15 000 lb. per sq. in.) and below those causing immediate failure, would have caused ultimate failure? Would a constant stress of any value above that amount and below the stress of immediate failure have done the same? Or, does it mean that the same curve would be repeated indefinitely for loads applied and released and producing stresses nearly as high as those which were found to cause failure? The curves seem to be alike in the characteristic of a gradual and even divergence upward from the straight stem, followed by a change in direction at a point representing a value of about 25 000 lb. per sq. in. for Columns Nos. 2 and 3, and 28 000 lb. for Column No. 5. Is it not probable that if the load producing the stress of, say, 25 000 lb. per sq. in., had been removed and replaced, there would have been a slight permanent set, and the second stress-strain line would have been straight and would not be changed by an unlimited number of repetitions of this load? Until the meaning of the stress-strain curve is satisfactorily explained, it seems to the speaker that proper working values for design cannot be derived from test values.

Assuming that it is desirable to place the safe working stress for calculated stresses not higher than one-half the maximum average value which could be repeated or maintained without failure or undue distortion, and assuming that this value was 25 000 lb. per sq. in. for the columns tested, a working value of 12 500 lb. would be derived. The common straight-line formula, $16\,000 - 70\,\frac{l}{r}$ would give a value of 12 700 lb. per sq. in. for these columns.

The speaker would again call attention to the very soft grade of steel in some of the column plates. Steel with a higher yield point will give a higher column strength, as shown by the tests of Messrs. Dagron and Waddell, as presented in the Report* of the Special Committee on Steel Columns and Struts, of this Society, and the test of the Quebec Bridge model column with revised latticing. The latter test was on a medium steel column, with a ratio of $\frac{l}{r} = 25$.

* *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 401.

The ultimate strength was 37 000 lb. per sq. in. on the Phœnix testing machine, the accuracy of which has been proven by Mr. Howard by a careful calibration. The stress-strain curve drawn by the speaker from the published test results for the Quebec Bridge model column, shows a practically straight line up to 30 000 lb. per sq. in. Mr. Carpenter.

In order to equalize more nearly the strength of the tension and compression members in structures, it appears to be necessary to vary the basic working stress or the material, or to combine the two methods. In the matter of material it would seem desirable to grade the harder melts into the compression members and the softer ones into the tension members and, at any rate, to avoid getting material softer than the normal into the compression members. The variation of the basic stress already appears in many specifications. From the results of these column tests and others recently published, and especially on the stress-strain lines for the same, it has been urged in some quarters that the basic compression stress be taken as low as 60% of the tensile stress. This seems to be unwarranted. The average carbon-steel eye-bar, which is the only form of large tension member of which there is a sufficient test knowledge, shows an average yield point of not much more than 30 000 lb. per sq. in., and it is necessary to use the grade known as medium steel to get that. The speaker would balance the additional safety from the strength beyond the yield point in tension members against the chance of a square break and the attendant disaster which could not occur in a compression member. Stress-strain records for tension members similar to those published for columns should be brought out and analyzed. The speaker has one for a carbon-steel eye-bar which shows a divergence from the straight line very similar to that of Column No. 5 of these tests, and at about the same points.

Engineers should not rush to the opposite extreme, in this matter of comparative working values; they need more tests like Mr. Howard's, of both compression and tension members, and more light on the significance of the stress-strain diagram.

J. S. Branne, M. Am. Soc. C. E.—This paper is interesting to the engineer who is trying to get at the inner workings of a large built-up column subjected to steadily rising stresses; and as the phenomena observed appear to have been recorded most accurately, and in such a way that one result will help to check another, many valuable lessons may be learned. Mr. Branne.

The steel used is quite mild, having never more than 0.28 of 1% of carbon, and the percentage of impurities is low. The ultimate strength appears to rise, generally, with the increase of carbon; but Melt 23 982 forms an exception, as in this the average of fourteen tests shows a lower value than for any other melt; its elastic limit, however, is high. It would be interesting if the author would give the probable explanation for this lack of uniformity in behavior.

Mr. Branne.

The action of the pin-plates under a moderate compression, not exceeding 9 000 lb. per sq. in. in the body of the column, is interesting as well as characteristic. The pin-plates, together with the column webs, take the applied load. At or near the pin it is evident that the unit stress, while not uniform over a plane normal to the axis of the column, as plainly indicated by the strains, is locally uniform in the pin-plates and column body, all receiving equally the stress from the pin, on the surface of the column, almost a smooth body. Receding from the pin, the unit stress in the column body will rise, due to the transference of the pin-plate stress through the rivets and by friction. At or near the last rivets in the pin-plate, this difference in unit stress will have its maximum value; hence the web from the pin to the end of the pin-plates is strained more than the pin-plates, and a small difference is detected, indicating an elastic motion of the pin-plates relative to the web.

Fig. 2, Plate XX, indicates a unit stress of 5 700 lb., and while the web cannot be stated by measurement in the same 10 in., yet in 30 in. it averaged 9 600 lb. When the test load was removed, this motion was reversed, and the set disappeared. It may be that when there are more and thinner pin-plates, this motion will be smaller, or the ideal condition would be approached of having just enough metal in the pin-plates at all points to make the unit stress the same as in the body of the column.

The local failure of Columns Nos. 3 and 4 might have been prevented by a long diaphragm, with its web parallel to the axis of the column and reaching close to the pin, thus minimizing the small bending moments in the two component parts of the column, preceding the dishing, as shown on Plates XXIII and XXIV.

Referring to the action of the lace-bars, the tests show that when the force is applied symmetrically with respect to the axis, or rather a force coinciding with the same, as in a testing machine, and with short and stocky columns, small lace-bars, simply balancing the component parts, so to speak, and preventing any first set, are quite ample, even for a column having two component parts of such magnitude as 45 sq. in. each.

Mr. Rights.

LEWIS D. RIGHTS, M. AM. SOC. C. E.—This Society is fortunate in securing a paper from a man so well versed in the science of testing metals as Mr. Howard. An examination of the records of column tests show that he was connected with all the important tests at Watertown Arsenal, and that he had charge of the tests of the eight large columns in connection with the calibration of the Phœnix machine. In a recent paper, "The Pittsburg and Lake Erie Railroad Cantilever Bridge Over the Ohio River at Beaver, Pa.,"* mention was made of the determination of stresses by gauge measure-

* *Proceedings*, Am. Soc. C. E., January, 1911.

ments. These measurements are being made, under Mr. Howard's direction, with gauges of the same type as those used on the columns described in his paper. Mr. Rights.

The reader's attention is attracted at once by the size of the columns. They are the largest ever tested to failure, having a cross-sectional area of 95 sq. in. Mr. Bland is to be commended for the broad spirit he has shown in spending a considerable sum of money to determine the character of the structures under his direction. He is also to be congratulated on being able to secure the services of Messrs. Buchanan and Howard, who conducted these tests, as their experience guarantees observations calculated to show a large number of interesting features. It is to be hoped that other large users of steel will follow Mr. Bland's example, and that much may be added to the data concerning steel columns.

Passing by many interesting features concerning the action of the details under stress, one important thing is the matter of local strains. The author mentions three conditions which affect the results, namely, cooling strains, initial strains due to straightening, and strains due to punching and shearing. Engineers have long recognized these conditions, and, in connection with columns, have endeavored to assign some arbitrary value which will take account of them. J. M. Moncrieff, M. Am. Soc. C. E., in a paper on "The Practical Column Under Central or Eccentric Loads,"* dwells quite fully on the subject.

According to the speaker's experience, about 95% of all the channels and 60% of all the angles which come from the average rolling mill have to be straightened after they are cold. Most of the work is done in a gag press, and it is evident that, in order to straighten a member, a small portion of it must be strained beyond the elastic limit on both sides. The side which was concave has been stretched in tension, and the side which was convex has been compressed, so that these two edges have been worked beyond the elastic limit, and have a considerable amount of initial strain. Now, when this material is placed in a column, it is also evident that the compressed side has comparatively little resistance to additional loads. This condition, of course, probably only applies to a thin section of the outside metal; nevertheless, it weakens the column by that amount.

The initial strains produced by punching the metal are not always recognized by engineers. It is well known by shop men that a long angle punched in both flanges will show a considerable lengthening, due to the punching, and, if this is not symmetrical, the piece will generally have to be straightened after being punched. The speaker has often had occasion to observe long channels, say, from 30 to 40 ft., which were intended for columns, and were punched throughout their length in both flanges. The punching tended to elongate the channel

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 334.

Mr. Rights. along the rivet gauge line. Naturally, the web, which contained the bulk of the material, was not elongated, and the result was that the channels were bowed to the extent of 2 or 3 ft. These channels had to be straightened before they could be assembled, and it is evident that there was considerable internal disturbance of the metal. Another feature of punching is the local disturbance in the vicinity of the punched hole. Some specifications call for the removal of $\frac{1}{16}$ in. of the metal around the hole by reaming. This, no doubt, does remove the badly injured material, and smoothes out slight differences between the punch and the die. If, however, one takes the trouble to rust the metal around the punched hole, the stress lines can be seen running in spirals away from it. It is probable that these stress lines do not extend into the metal very deeply, but they indicate that engineers are not dealing with an ideal material, and that the strength of columns is based on a number of such conditions, which are not always fully recognized.

Fig. 6 shows the stress-strain curves for three of the columns. It will be noted that the author has recorded a seeming departure from the line of proportionality at about 20 000 lb. per sq. in. He seems to think that this is due to the various local strains which he has mentioned, and which probably occur in all columns. The speaker agrees with Mr. Howard, and believes that if extremely delicate instruments could be secured, these local adjustments would show at even lower values than 20 000 lb. He does not consider, however, that this may be interpreted to mean a complete departure from the elastic conditions at these low stresses. Probably, as the minor local conditions in the component parts are adjusted, those portions which were in tension take on additional loads and relieve the compressed parts. The speaker believes that if a new set of gauge measurements were made on the column after it had been stressed up to 25 000 lb. per sq. in., the results of the local failures would no longer be apparent, and the column would show a strict proportionality up to about 27 000 or 28 000 lb., which, as indicated on Fig. 6, seems to be the yield point.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E.—The speaker desires to express his sense of obligation to the author and to Mr. Bland for the information contained in this paper.

The paper is important for several notable reasons. It throws light upon the relation of large-sized tests to small tests. In the past many tests have been made on small specimens, but the number on large specimens is so limited that every additional one helps.

Another point which, of course, is very prominent in the paper, is the extreme refinement of observation which characterizes it. It is doubtful whether as many measurements of deformations have ever been made as were made in this particular instance.

To be sure, many of the results bear out what might have been

expected. On the other hand, there are some which are quite surprising, and rather difficult to explain; but they all tend to show one important fact which is apt to be overlooked, namely, that there are parts of a built-up member which are stressed far beyond the average stress. Engineers are apt to think of the stress applied to the area of the column section as being uniform throughout. Considering the results of these experiments, however, they must bear in mind, that the distribution of stress is not uniform, but that if the total stress is divided by the area, the average result by no means represents the maximum. Mr. Worcester.

Another important point is shown by the stress-strain diagrams, which clearly indicate the low stress at which the proportionality between stress and strain seems to be lost. The exact explanation of this may be clearer after more tests have been made, though Mr. Howard's analysis undoubtedly helps to explain it.

It is sincerely to be hoped that the author's example will be followed, and that others who have the opportunity to provide full-sized tests will see their advantage to the Profession.

Horace E. Horton, M. Am. Soc. C. E. (by letter).—In discussing this paper the writer wishes to express to Mr. Howard and his colleagues of the Bridge Department of the Pennsylvania Lines West of Pittsburg, his obligation for their contribution of the very large proportion of all the research work, of record, bearing on this subject. Mr. Horton.

On another occasion, in discussing the compressive member, the writer said:

"It is subtle and illusive, and may side-step."

"It is composite, and not homogeneous."

"No law is written as to the proper relation and proportion of its constituent parts."

Here he will add: There is no clear and definite knowledge of how to get the greatest efficiency out of the material used in a wrought-metal, composite, compressive member.

Lest the writer should repeat himself, it seems best to refer to his discussion of the paper by J. R. Worcester, M. Am. Soc. C. E., on "Safe Stresses in Steel Columns."*

Mr. Howard's tests show ultimate strength in compression of only one-half the ultimate strength in tension of the same class of material. If it is assumed that the elastic limit in tension is one-half of the ultimate strength, and that the rational working load is one-half of the elastic limit, then the ultimate working load is one-quarter of the ultimate tension. If the ultimate strength in compression is one-half of the ultimate strength in tension, and the elastic limit in compression is three-quarters of the ultimate strength in compression, then the ultimate working load in compression would be three-eighths

* *Transactions*, Am. Soc. C. E., Vol. XLI, pp. 178 to 186.

Mr. Horton.

of the ultimate strength in tension; that is, the working load in compression is three-quarters of the working load in tension.

In connection with Mr. Howard's paper, it seems well to call attention to the Progress Report of the Special Committee on Steel Columns and Struts,* showing results of tests on 263 full-sized members. These, with Mr. Howard's five tests added, make 268 in all:

184 on riveted iron members;

52 on steel homogeneous sections 25 $\frac{l}{r}$ or more (pipe and H-figures); and

32 on riveted steel members.

The ratio of the ultimate compressive strength (also the elastic limit on pipe, H, and riveted-steel members), shown by these tests, to the ultimate tensile strength of the material, is indicated in the diagram, Fig. 8.

The 184 tests on riveted iron members are in a sense obsolete, and with the 52 pipe and H-sections—236 tests in all—are useful as an index of the decreasing unit value in compression as $\frac{l}{r}$ increases. The 52 homogeneous tests of pipe and H-sections are further useful in showing the probable full efficiency of the steel compressive member. It is interesting to note that the pipe and H-sections, in plating, followed essentially the same line.

Of riveted-steel compressive members, built up of parts riveted together—composite, not homogeneous—there are only 32 tests, from 25 to 97 $\frac{l}{r}$.

The average ultimate strength of compressive members, as shown by Fig. 8, 25 to 97 $\frac{l}{r}$, for pipe and H-sections, is from 75 to 50% of the ultimate tensile strength; and it is from 60 to 43% for riveted sections.

The average elastic limit of compressive members, as shown by Fig. 8, relative to tensile strength, for pipe and H-sections, is from 52 to 47%; and it is from 45 to 20% for riveted sections.

This is both interesting and instructive, because it shows, for riveted members, only 65% of the elastic limit of the homogeneous ones, that is, 65% efficiency—35% loss in the composite riveted member. Considering that engineers, from the nature of their work, are obliged to use riveted members, surely it is desirable to raise the efficiency above 65 per cent. How is it to be accomplished?

Referring particularly to the five tests by Mr. Howard, it is an interesting speculation to query what would have developed if:

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* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 401.

Mr. Horton.

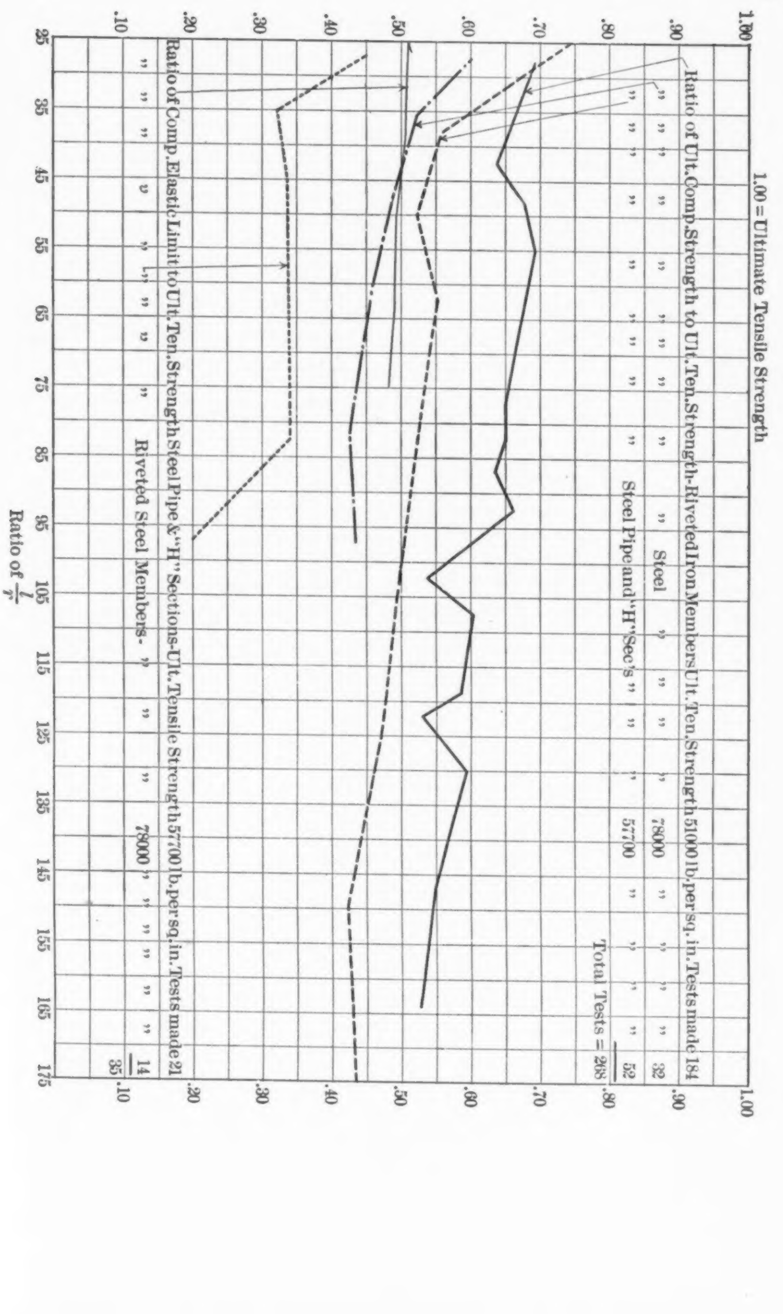

FIG. 8.

Mr. Horton.

(a) The members had been exactly as tested, except that twice as many $\frac{7}{8}$-in. rivets had been used;
(b) Same as (a), but with twice as many $\frac{7}{8}$-in. rivets (four times as many $\frac{7}{8}$-in. rivets as in the member tested);
(c) If the members had been exactly as tested, with same number of rivets, but with the rivets increased to $1\frac{1}{4}$ in. in diameter;
(d) Same as (c), but with double the number of $1\frac{1}{4}$-in. rivets;
(e) If the compressive members had been built of four 6 by 6 by 1-in. angles and two 28 by 1-in. plates, without diaphragms, $\frac{7}{8}$-in. rivets, the same tie-plates, the same lattice, in fact, the columns of the same weight as those tested;
(f) The same as (e), but with twice as many $\frac{7}{8}$-in. rivets;
(g) The same as (f), but with double the number of $\frac{7}{8}$-in. rivets (four times as many $\frac{7}{8}$-in. rivets as in the member tested);
(h) Same as (e), with $1\frac{1}{4}$-in. rivets, the same number as in the member tested;
(i) Like (h), but with double the number of $1\frac{1}{4}$-in. rivets.

This would give nine modifications, added to the test already made, making ten; with five of each class, equaling 50; and 20 changes of $\frac{l}{r}$, extending from 25 to 200, would require 1 000 test pieces and tests.

If the length, breadth, thickness, and all component parts of these pieces were of one-eighth size, each unit would weigh $\frac{1}{512}$ of each member described by Mr. Howard, and it can be safely concluded that the cost of the material for the 1 000, ready for the testing machine, would be less than that of the five units tested.

The $\frac{1}{8}$-dimension test member representing (e) would be made up of four $\frac{3}{4}$ by $\frac{3}{4}$ by $\frac{1}{8}$-in. angles, two $3\frac{1}{2}$ by $\frac{1}{8}$-in. plates, and $\frac{7}{64}$-in. rivets. For (h) $\frac{5}{32}$-in. rivets would be used.

It is unfortunate that the Special Committee on Columns and Struts has gone on record as indicating a want of confidence in "Laboratory Tests." What is a laboratory test? The engineering press has called attention to a design of a structure wherein a compressive member having a cross-section of 2 037 sq. in. is contemplated. Mr. Howard's 90-sq. in. sections are essentially one-fifth of this. In some of the tests reported by the Committee the members are less, in each of the three dimensions, than one-third of the dimensions of other test members under consideration; hence a member of $\frac{1}{3}$ dimension is not a laboratory test specimen because the Committee has recognized it.

The smaller cross-sections in the Committee's report bear a relation to the proposed 2 037-in. section of quite one-sixteenth for three dimensions, that is, the Committee has recognized as rational-sized

Mr. Horton.

tests, members so slight, as compared with the member of 2 037-in. cross-section, that the weight of a unit of each, with the same $\frac{l}{r}$, is as 1 to 4 096. The writer has suggested a $\frac{1}{8}$ dimension of Mr. Howard's 90-in. section. This would be approximately a $\frac{1}{3}$ dimension of the smallest size the Committee has recognized.

Does the Committee believe that there is concealed about any engineer, knowledge of the compressive member equal to what might be expected to develop from 1 000 so-called laboratory tests of $\frac{1}{8}$-dimension pieces along the lines indicated above?

From what source have engineers gathered their knowledge of the compressive member? Large numbers of laboratory tests have been made on small solids and small pipe, all homogeneous sections, which are largely the source of all present knowledge as to the compressive member. The writer's appeal is that the research be extended by including tests of large numbers of composite, riveted, wrought, compressive members, which may be handled in a 100 000-lb. testing machine.

The engineering schools have testing machines of sufficient capacity to handle the specimens herein indicated. They have ambitious professors and zealous students. With slight encouragement, on the part of the Committee, a series of tests could be carried out which would give definite knowledge, and, the writer believes, would indicate the way in which the efficiency could be raised to more than 65 per cent.

Mr. Talbot.

Arthur N. Talbot, M. Am. Soc. C. E. (by letter).—These tests are noteworthy in many respects, particularly in that a study was made of the action of the column in its different parts at loads below the ultimate. Tests to destruction may not of themselves have much meaning. The author is also to be commended for his interesting and valuable discussion of the data of the tests.

The writer is not entirely satisfied with the conclusion that the elastic limit of the material in tension has a controlling influence on the ultimate strength of the column. Throughout the paper the author seems to assume that length of column does not affect the strength or modify the distribution of stress for any load. Although the trend of evidence in available tests is that, within ordinary limits of length, column action does not follow the law which is assumed in the derivation of the old column formulas, yet the consensus of experimental data indicates that for ordinary lengths of column there is a noticeable decrease in strength with an increase in length or slenderness ratio. Tests made at the University of Illinois also indicate that the effect of length on deformation is greater at the higher than at the lower loads. How much of the loss in strength is due to increased opportunity for non-homogeneity of materials, non-straightness of pieces, and non-integrity of section cannot be told. However, from the ex-

Mr. Talbot. perimental data which are available, it may be expected that a column of the length of No. 5 will be weaker than one of half this length.

Elastic limit of material is an elusive thing, not easily defined, and the finer the measurement the lower the stress at which set and lack of proportionality of stress to elongation may be found. The effect of exceeding the elastic limit of steel has not yet been determined. To exceed the yield point in any part of a structural member is a more serious matter; the distribution of stress is at once modified, and a larger stress is laid on other portions of the cross-section. The measurements in the tests were not numerous enough or sufficiently distributed over the length and cross-section of the column to determine how much the maximum local stresses were in excess of the average stresses; but, although the column tested was of very stocky section, it seems almost certain that the average stresses must have been greatly exceeded in places, even before the yield point of the metal was reached anywhere. The passing of the yield point locally may account in part for the deviation of the stress-strain curve from the straight line in Fig. 7. Such a variation from the straight line is to be found in the test of any built-up column. To the writer it seems that the evidence of the tests, taken in connection with phenomena observed elsewhere, indicates that the yield point of the metal must have been exceeded at different points in the column at loads below the ultimate load, and that the fact that the average stress at failure was about the same as the elastic limit of the softest metal is only a coincidence.

The author's comment, in connection with the increased intensity of stress in the vicinity of the pin bearings, that "details of construction generally exert an influence on the behavior of the member, and not infrequently modify the ultimate resistance materially," is worthy of emphasis. Too often designers think the problem is solved by the mere provision of sufficient metal, without any consideration of the manner in which the stress may be transmitted and distributed. Before a pin-plate can transmit stress to the web-plate, there must be a difference in deformation in the two plates, particularly if there is no movement of the pin-plate over the web-plate. If there is slip (and if we are to judge from the tests of the riveted joints, a movement must have occurred long before the full loads on the columns were reached), then there will also be a deformation or detrusion of the rivets to transfer stress from plate to plate, and this transfer of stress will take place at those portions of the pin-plate well away from the pin. The tests indicate that this movement of the plates and detrusion of rivets were present. They would also seem to be the cause of the difference in measurements, rather than any bending of the pin.

The writer is glad that the author recognizes the effect of time on the action of a compression piece stressed locally beyond the yield

point. If a longer time were given in tests of short pieces, quite different data would be obtained. Mr. Talbot.

It would have been interesting if systematic measurements had been made at various points along the length of the column, and on all sides of the component members, to ascertain the amount of local wrinkling or buckling and also to determine the stress in all lattice bars. It would also add to the completeness of this excellent paper, if the author would give a record of the permanent set found in the columns.

JAMES CHRISTIE, M. AM. SOC. C. E. (by letter).—These columns were carefully designed, and, doubtless, carefully constructed; and the minor disturbing features, referred to by Mr. Howard as due to unequal cooling, cold-straightening, etc., probably would not reduce their resistance seriously as a whole. Mr. Christie.

There are reasons for believing that when $\frac{l}{r}$ equals about 100, the resistance to compression of a column, per unit of its section, should be approximately equal to the elastic limit of its material, and that columns having the same ratio of length to section as those tested, should develop a maximum resistance considerably exceeding that of the elastic limit of the material. As the recorded resistance of these columns fell a little below the said elastic limit, it would be interesting to know how much, if any, of this was due to insufficient area of pin bearing. Mr. Howard intimates that this may have been a source of general weakness. Under the maximum compression the bearing areas for the pins were stressed much higher than the elastic limit of the material, and probably nearly as high as its ultimate tensile strength.

This excessive burden was indicated by the failure of the reinforcing pin-plates and the local disturbance at these ends, and probably contributed toward the general failure of Column No. 5. As Mr. Howard well remarks:

"Regularity of action is limited to stresses within, and ceases at, the elastic limit, and when over-straining of the metal occurs, locally or in general, the strength of the column is menaced."

N. R. MCLURE, ASSOC. M. AM. SOC. C. E. (by letter).—Each intelligently made test on large-sized, built-up, steel, compression members adds to the all too scant knowledge of their action; and the publication of the results of these tests in order that interested engineers may study them and draw conclusions therefrom, cannot be too highly commended. The author has done a great service in adding to the small stock of information on this subject. Mr. McLure.

In the five tests which form the basis for this paper, the average ultimate strength of the columns, as shown in Table 1, is about 29 500 lb. per sq. in. The average elastic limit of test specimens for angles and plates composing these five columns, taking the value of

Mr. McLure. the angles as 31% of the total cross-section of each column, is about 32 300 lb. per sq. in. As the elastic limit in a test specimen will usually run about 10% higher than that of the full-sized piece from which it is cut, the average value for the elastic limit of the material entering into these columns may be taken as about 30 000 lb. per sq. in., which is practically equal to the average ultimate strength of the columns. In that point, these tests agree with those on other full-sized columns which the writer has noted during the past three years, and, in fact, it has almost proved a rule without exception that the ultimate strength of any latticed steel column, with properly designed details, and a ratio of $\frac{l}{r}$ of less than 60, is about equal to the elastic limit of the metal composing the cross-section of that column. It is evident, from several remarks in the paper, that the author has recognized the influence of the elastic limit of the metal on the ultimate strength of the column.

The number of points covered in the tests on these columns, and the opportunity given, by the numerous measurements of local strains, to study the detailed action of the load throughout the member, make them doubly interesting, and emphasize the desirability of establishing a standard for conducting tests on full-sized compression members, so that the necessarily few tests made from time to time, as occasion requires, may be grouped and tabulated for comparison. It seems to the writer that the era of full-sized tests on steel compression members is just beginning. With the new testing machines now in course of completion, the apparatus available for making tests will stimulate interest in them, and a continuously increasing study of the action of built-up steel compression members may be expected. It seems, therefore, that the Society could further the value of such future tests in no better way than by recommending a general standard method for conducting them, including the desirable observations to be made and other details which, if adhered to, would bring all tests to a suitable basis for comparison. By so doing, doubtless in a few years the old column formulas, based on the ratio, $\frac{l}{r}$, might be replaced, or at least supplemented by more rational ones applicable to the various types of built-up steel compression members.

The author's observations in regard to local strains are most interesting, particularly the agreement of the actual with the theoretical ratio of lateral expansion to direct compression.

Each new test of this sort only emphasizes the important and modifying influence of details of construction on the final result, and this should serve as an incentive to all designers to spare no effort in working out details which, without an atom of doubt, will develop the full compressive strength of the member.

Mr. McLure.

The influence of initial strains, referred to by the author, is one which cannot readily be taken account of in design, except by allowing a sufficient margin of ultimate strength over working load. Annealing may at times be resorted to, but the elastic limit of the metal is thereby reduced, which reduction might be more serious than the presence of local zones of over-strained metal. Punching, shearing, straightening shapes while cold, and other manipulation of the steel during fabrication and erection—including the use of drift-pins—are all necessary evils in the present state of the art, the effects of which will always be uncertain. By numerous tests, similar to those recorded in this paper, however, we may hope gradually to increase our knowledge and better our results, and the writer wishes again to emphasize the importance of conducting such tests according to a definite standard, and of publishing the results for the benefit of the Engineering Profession.

Mr. Chew.

R. S. Chew, Assoc. M. Am. Soc. C. E. (by letter).—This is a most valuable paper, as it presents in detail the actual strains in compression members under load, and emphasizes the sources of their weakness.

The writer believes that the column formula will eventually take account of the imperfections which really exist in built members, namely:

First.—Initial stresses in material due to manufacture,
Second.—Variation in strength of component parts of section,
Third.—Crookedness of component parts,
Fourth.—Crookedness of whole member,
Fifth.—Local stresses due to details and shop work,
Sixth.—Accidental eccentricity,
Seventh.—Deflection caused by the foregoing imperfections.

These items, with one or two exceptions, have been covered by Mr. Howard's tests.

Referring to the variation in strength of the component parts of the section, it is seen that in Table 5 the author has given the elastic limits of the shapes composing the section (it is to be regretted that he did not give the actual location of these various pieces in the section). In Column No. 5 if it is assumed that the pieces of lower elastic limit are grouped in one half-section (a condition which might exist), the resisting axis is 0.178 in. removed from the center of gravity. Column No. 5 is of excellent proportions, and the eccentricity due to this item is small; still, the idea that a perfectly straight compression member with the load applied at the center of gravity will develop a definite bending moment, independent of $\frac{l}{r}$, is important. It can be readily seen that a latticed channel column with a 25% variation in elastic limit would have a high eccentricity.

Mr. Chew.

The writer is heartily in accord with the author in the statement:

"If it is accepted that the elastic limit has a controlling influence on the ultimate strength of a column (as it is believed to have), then variations of from 25 to 27% in this value, as witnessed in the tests of the metal of these plates and angles, would overshadow those considerations which find expression in empirical formulas for strength, and take no account of such features."

The suggestion of remedy in designing work, however, would be to assume the existence of a variation in the elastic limit between the two halves of a section, locate the resisting axis, and provide for this bending moment.

It is curious that the author makes no note of the condition of the specimens with reference to crookedness. It is also strange to find that standard specifications simply require that such members shall be straight and true. It is a fact, however, that they are not straight, and the writer believes that a limit of departure from straightness should be specified. Those in charge of one of the leading testing bureaus state that commercial columns, 30 ft. long, fabricated under the best conditions, have very seldom less than $\frac{1}{4}$ in. of kink or bend in them. The fact that Column No. 5 failed at the one-third point, instead of at the middle, leads to the belief that there may have been a kink near that place.

The item, local stresses, is interesting. The author's investigation shows the importance of good details. However, the statement that there is a contractile force of 30 000 lb. per sq. in. for riveting is not in accord with the writer's experience, for he has noted cases of heavy structural members, inspected and passed at the shop by competent men, in which a number of rivets have worked loose during shipment. This would seem to indicate that the rivet in tension was strained beyond the elastic limit, so that the contractile force would be much less than the author has assumed.

The writer understands that the investigation with reference to the stresses in lattice bars was made at the middle of the column length. Although these bars may help to distribute the load, to a limited degree, and also tie the sections together, yet one of their main functions is to carry the transverse shear produced by any bending moment in the column, which shear is zero at the middle. It would be interesting to know how these stresses vary toward the ends.

The writer believes that the investigations described in this paper will be a great aid in the development of a column formula. It may or may not be agreed that the six items cited by the writer produce the mysterious bending moment in columns; however, if a limit for the variation in the elastic limit—degree of straightness—can be defined, it is believed that a formula including the moment caused by these items would approximate to working conditions.

GEORGE N. COLE, ASSOC. AM. SOC. C. E. (by letter).—One of the discussors has mentioned the desirability of more delicate instruments for the measurement of changes in large steel columns under load. The writer has noted the description of Mr. Howard's application of the micrometer screw, and believes that small mirrors might be used in testing large bridge members. Some of these mirrors might be attached directly to the column and others could be set up on a separate axis and made to rotate by any motion of the column. Mr. Cole.

By this means the bending or twisting of the member could be observed in a state of rest and under load, it being understood, of course, that a fixed source of light is furnished for the mirrors, and that graduated scales are placed at the right distances to magnify properly all motions of the beam and render them calculable.

The writer understands, of course, that this method has not the continuous possibilities of Mr. Howard's ingenious centering for his micrometer screw, but it is thought that it might have an application in certain of the studies of column tests, and for movements under load.

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PAPERS AND DISCUSSIONS

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SINKING A WET SHAFT.

Discussion.*

By Messrs. Mason D. Pratt, Charles B. Buerger, L. White, and H. M. Hale.

Mr. Pratt. Mason D. Pratt, M. Am. Soc. C. E. (by letter).—Two of the difficulties mentioned by Mr. Hogan are of interest to the writer because they correspond exactly with conditions met in driving an intake tunnel, in 1906 and 1907, from the power plant of the Central Pennsylvania Traction Company, at Harrisburg, Pa., to the Susquehanna River, a distance of 1 100 ft. This tunnel was driven entirely through limestone formation, and was from 5 to 20 ft. below the river level. In sinking the midway shaft and driving the headings for 200 or 300 ft. each way from the shaft, H_2S gas was encountered in such quantities as to cause serious inconvenience to the men, and for a time artificial ventilation was necessary. The gas affected the eyes, and the men required frequent medical attention; occasionally their eyelids were swollen so much that their eyes were entirely closed for a day or two. The tunnel passed directly under the yard of the Central Iron and Steel Company, and near two blast furnaces, and the gas was attributed to the furnace-slag filling which was used extensively around the yard of the steel works. This explanation was rational and satisfactory, but Mr. Hogan's experience on the Rondout shaft, where similar conditions did not prevail, would make it appear that the gas might have come from other sources.

At several times large volumes of water were encountered, the workings were flooded, and the pumping difficulties became most discouraging. Although the central shaft was only 45 ft. deep, and the

*This discussion (of the paper by John P. Hogan, Jun. Am. Soc. C. E., published in *Proceedings* for March, 1911, and presented at the meeting of May 3d, 1911), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

writer did not have the additional difficulties of high-head pumping, as at Rondout, yet shaft pumps were necessary, and the troubles were not overcome until reciprocating pumps were abandoned entirely and there had been installed three No. 4 Emerson pumps, which at times delivered from 1 500 to 2 000 gal. per min. Grouting of seams was resorted to; the most serious case occurred when one heading had advanced 400 ft. from the shaft, and the seam was successfully closed with grouting forced through a 2-in. line from the top of the shaft, with an ordinary hand-power grout-pump, saw-dust being mixed with the grout. Mr. Pratt.

One other difficulty on this work, which nearly caused its abandonment, was a large inflow of hot water from Paxton Creek, under which the tunnel passed. A short distance down stream from the center line of the tunnel, the creek had been dammed, and the pool was used as a reservoir for condensing water in the Traction Company's plant and for cooling water at the blast furnaces. This point was passed during the summer, when the flow in the creek was practically nil, and the temperature of the water passing into the tunnel was, at times, as high as 126°, so that the men had what was probably their first experience with a Turkish bath. This difficulty was not overcome until the opening from the creek was located and surrounded with a coffer-dam.

Charles B. Buerger, Assoc. M. Am. Soc. C. E. (by letter).—This paper shows that, in this work, the greatest obstacles to progress were: the difficulty of keeping the unwatering pumps in satisfactory working condition, when operating against a head exceeding 200 ft., and the large number of pumps with their connections needed in the small space available. For instance, at a depth of 300 ft., for a delivery of 725 gal. per min., ten pumps were used, although, ordinarily, this quantity could be delivered by two of the larger Cameron sinkers. Mr. Buerger.

The objectionable pulsations in the discharge piping were probably the result, not of the higher pressures, but of the increased length of the discharge column, with the correspondingly increased inertia of the water in this pipe, and greater water-hammer. Pumps of the shaft-sinking type are usually fitted with very small air chambers. It would be an interesting experiment to fit a chamber of considerable capacity on the discharge of such a pump. Such an air chamber could be designed to occupy a very narrow width, and be long enough to give the required capacity. For severe conditions an additional chamber could be placed in the middle of the length of the discharge pipe.

Considering the reduction in capacity of both pumps and all their piping and connections, as found in actual service, and the inconvenience caused by the fact that they occupied so much space, the use of such air chambers would seem to have a valuable field; they have

Mr. Buerger.

been used successfully in the penstocks for water-wheels working under extreme conditions of length of supply pipe and variable load; only a service trial with the system, however, could determine the facts.

Mr. White.

L. WHITE, ASSOC. M. AM. SOC. C. E.—The author has presented the eventful history of Shaft No. 4 of the Rondout Siphon so fully that it is difficult to add to the paper, or to dissent from what he has stated, or enlarge upon his sound conclusions.

It is perhaps well to re-state that no structural weakness of the rocks at this point gave any trouble, the case being simply that of a few porous layers heavily charged with water and H_2S gas. The Binnewater sandstone has a strong resemblance to the sand rocks of the oil region, but, instead of yielding oil and gas in great quantities, it yields only valueless sulphur water and gas, unless, indeed, some day it is bottled as a new brand of Hunyadi or Pluto water. In the speaker's opinion, the sulphur gas gave the most serious trouble, for besides making it hard for men to work in the shaft, it forced the water out under pressure and maintained the large and high flow encountered.

As one of the superintendents well stated, "It would be hard to find another spot yielding water so persistently as Shaft No. 4." One of the lessons to be learned from this work is to place as little reliance as possible on vertical sinking pumps hung in a shaft, and to use freely horizontal pumps in rings or chambers, only utilizing sinkers to pump from the bottom of the shaft to the lowest chamber. One of the most experienced shaft men even used horizontal pumps on the bottom, slinging them to chains in order to raise and lower them by the main cable at the time of shooting.

As described by Mr. Hogan, the grouting was of great aid in sinking the shaft, and was very successful, particularly in connection with a heavy seam at first encountered. It is not easy to determine whether or not the grouting of the diamond drill holes could cut off much water, but it seems to have been effective, for, when the side of the shaft at the large chamber was drilled, large quantities of water were encountered in the hole within a few feet, but the shaft itself did not seem to be particularly wet at that level.

If this had been a permanent shaft, it probably would have paid to place a concrete lining in the wet zone to cut off the water. This could have been readily done by pumping the water from the back of the concrete lining, as was done on a faulted zone at Shaft No. 5, where 125 gal. per min. were cut off. In this way the pumpage could have been cut down to almost the quantity found at the bottom.

It is interesting to recall that the troublesome layers in the shaft also proved troublesome in the tunnel just north of the shaft; that they yielded nearly 2 000 gal. per min. at one time for a stretch of scarcely more than 100 ft.; and that the grouting, unlike that in the

shaft, was not successful in cutting off any flow, the porous rock being so broken up by faulting that the cement failed to cut off the water. Mr. White.

Eventually, the tunnel dried out the shaft, so that the chamber pumps were removed to the bottom, at which time there were four separate discharge lines in the shaft—two of 10-in., and two of 12-in. Eight air-operated pumps, of the Cameron reciprocating type, discharged into them. Three of these had a capacity of 450 gal., three of 350 gal., one of 200 gal., and one of 330 gal., making a total air-pump capacity of 2 930 gal. per min. against a 500-ft. head. In addition to these pumps three 6-stage Worthington centrifugal pumps were installed later. These were electrically operated, and their capacities were 560, 600, and 700 gal., respectively, making a total of 4 790 gal. per min. against a 500-ft. head.

Very wisely, a concrete bulkhead was built, having an opening closed by a heavy wooden door. Various pipes for air, water, etc., were embedded in the bulkhead when the concrete was placed. When an explosion of the dynamite magazine at the top of the shaft wrecked the transformer house, putting the electrical pumps out of commission, the door was closed, shutting off the water in the heading, and the remaining leakage through the bulkhead was handled by the air-pumps. This probably saved the shaft from flooding. The ground-water at Shaft No. 5 has been lowered about 500 ft., and the tunnel adjacent is making about 1 350 gal. per min.

The bad ground at Shaft No. 5 revealed the character of the rock, so that excavation at this point was delayed about a year. Even with this delay, the T. A. Gillespie Company expects to complete the whole contract on time. This is due to the great energy shown at the beginning, when the work was pushed while Shaft No. 4 was delayed. Had it not been for the energy and resourcefulness displayed by the contractor, the work might still be far from finished, and, instead of telling how the job was done, the engineers might be looking for some one to tell them how to do it.

H. M. Hale, Jun. Am. Soc. C. E.—This paper is a valuable contribution to engineering literature on difficult shaft-sinking. It describes what is without doubt the most trying piece of work which the engineers of the Board of Water Supply have encountered, and it is a matter of congratulation that the resourcefulness of the contractor accounts, in a large measure, for the opportunity to discuss the history of this shaft. Mr. Hale.

Shaft-sinking pumps will pump, but, under high heads, discharge lines with flexible connections act like the weakest link in a chain. The author fully points out their weaknesses in that respect, and the lesson to be learned is embodied in the successful application of concrete rings and chambers where the duties of the sinkers can be transferred to horizontal pumps with rigid connections.

Mr. Hale. Mr. White's discussion touches on the general conditions met in the tunnel driven north from Shaft No. 4, in which the strata of porous rock corresponding to that in the chamber were struck, resulting in an increased flow in the tunnel and an absolute loss of water in the shaft. It is fortunate that the intimate relation between the chamber and the porous zone, penetrated later in the heading, existed, on account of its bearing on the tunnel driven from Shaft No. 3 southward through the Helderberg limestone. The maximum flow into the tunnel (2 000 gal. per min.) lowered the water level in the whole region more rapidly than would have been the case had the greatest amount of pumping been confined to the water flowing into the chamber alone.

In the tunnel driven south from Shaft No. 3 approximately 1 000 ft. were in a zone of overthrust faults and vertical fractures, extending from Station 587 to Station 597. In every case the fractures running diagonally across the tunnel showed water-worn surfaces and cavities which had been enlarged by solution at a time when the region was above sea level. The result was that these cavities, most of which extended far above the tunnel roof and to unknown distances in both directions, were completely filled with a "residual" clay, which contained absolutely no grit. In one wide seam there was found a bed of clean sharp sand, with a small proportion of pebbles. The bed thinned out at the edges where it pierced the surrounding clay.

In the tunneling these seams dropped their contents into the heading, but only in one case was there any flow of water into the tunnel. The first clay pocket containing sand gave a flow of about 200 gal. per min. Before the subsequent caves had been opened up, the chamber at Shaft No. 4 ran dry, causing the flow in the tunnel to decrease to less than 100 gal. per min. for its whole length. No water accompanied the inflow of soft clay after the first encounter; therefore only a small proportion of clay in the seams had to be taken care of. The seams cleared themselves wholly only for that portion of their length covered by a little more than the width of the tunnel. Beyond this, they held the clay contained in them and gave very little trouble thereafter.

Some of these caves are of considerable size, the largest being about 5 ft. wide, 20 ft. high, and extending for about 30 or 35 ft. to one side of the tunnel.

With such conditions, therefore, the results might have been far worse had the ground-water not been lowered below the level of the tunnel by the large inflow of water in Tunnel No. 4 North, which, at the foot of Shaft No. 4, is about 145 ft. lower.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE BOWERS CALDWELL, M. Am. Soc. C. E.*

DIED MARCH 31ST, 1911.

George Bowers Caldwell, son of Lucia Morgan and Lewis H. Caldwell, was born in Lowell, Mass., on April 11th, 1863, where he received his early education in the public schools. At the age of seventeen, he became an apprentice of the Lawrence Manufacturing Company, of Lowell, serving his time in the shops and drafting room. At the end of his four years' apprenticeship he remained with that company, as Assistant to the Master Mechanic, until 1889, when he accepted the position of Master Mechanic of the Washington Mills Company, of Lawrence, Mass., where, for four years, he was in charge of the drafting room, as well as the construction and repair work.

In 1893 Mr. Caldwell entered the employ of Westinghouse, Church, Kerr, and Company, and retained his connection with this firm until his death. During this period he was in responsible charge of designing and constructing many works of engineering interest, among which were the following:

The mechanical and electrical features of the South Terminal Station, at Boston, Mass.; the Pittsburg Terminal of the Pittsburg and Lake Erie Railroad; the railroad shops of the Pittsburg and Lake Erie Railroad at McKees Rocks; and the construction of the Kingsbridge Power Station of the Third Avenue Railroad. He also supervised the design and installation of the electrification of the Long Island Railroad, including the large power-house at Long Island City. He was in charge of the design and construction of the mechanical and electrical features of the Pennsylvania Terminal Station, in New York City, but was obliged to relinquish it just before it was completed. In all his work he showed engineering qualifications of a high degree.

In 1889 Mr. Caldwell married Mary E. Fraser of Lowell, Mass., who, with four children, survives him.

After the completion of his work in connection with the South Terminal Station, in 1899, Mr. Caldwell moved to Yonkers, N. Y., where he resided until his death.

He took an active interest in civic and social affairs, and was well and widely known in the neighborhood of his home, being an active worker and Trustee in the Baptist Church of the Redeemer and a Charter Member of the Nappeckamack Club, of Yonkers. He was also

* Memoir prepared by George B. Francis, M. Am. Soc. C. E.

a Member of the American Society of Mechanical Engineers, of the Grecian Lodge, A. F. & A. M., and of the Mt. Sinai R. A. Chapter, of Lawrence, Mass.

Mr. Caldwell was a man of unwavering integrity and of great energy, inspiring perfect confidence and respect in all those with whom he dealt. He was not only a loving, but a wise parent, and his death leaves a void, not only in the hearts of his family, but in the hearts of those for whom he worked and who worked for him.

Mr. Caldwell was elected a Member of the American Society of Civil Engineers on April 1st, 1908.

JOHN FRANKLIN HINCKLEY, M. Am. Soc. C. E.*

Died February 20th, 1911.

John Franklin Hinckley was born in Boston, Mass., on December 13th, 1849. His parents were John Loving Hinckley and Emeline Hinckley. He received his education in the public schools and at the English High School of Boston.

His father was engaged in the railroad business in the West, and in July, 1868, Mr. Hinckley joined the Engineering Corps of the Atlantic and Pacific Railroad (now the St. Louis and San Francisco Railroad), and was Assistant Engineer on its construction from April, 1869, until May, 1870, and Assistant Engineer on surveys until July, 1871, at which time he entered the service of the Missouri Pacific and was employed on the construction of its Lexington Branch.

From January 1st, 1872, to August, 1873, Mr. Hinckley was Assistant Engineer on the Indiana and Illinois Central Railroad, and from August, 1873, to December, 1875, he was employed as Chief Engineer on the Havana, Rantoul and Eastern Railroad. In December, 1878, he was appointed Division Engineer in charge of the construction of 50 miles of the Chillicothe, Brunswick and Omaha Railroad. In July, 1879, he entered the service of the St. Louis and San Francisco Railroad, remaining in the Company's employ as Division Engineer and Principal Assistant Engineer until January, 1887, when he was appointed Chief Engineer of the St. Louis, Arkansas and Texas Railway (now the St. Louis Southwestern Railway), and had charge of the construction of branches to Shreveport, La., Hillsboro, Tex., Little Rock, Ark., Fort Worth and Sherman, Tex., until July, 1888, when he resigned.

In 1888 and 1889, Mr. Hinckley was Assistant to Robert Moore, Past-President, Am. Soc. C. E., on the Merchants Bridge Terminal

*Memoir prepared by C. D. Purdon and F. G. Jonah, Members, Am. Soc. C. E.

Railway in St. Louis, Mo., and from 1889 to 1894, he was associated with Messrs. A. W. Scott and J. H. Hedges in contracting work and in a quarry. During this time he did a great deal of work for the Kansas City, Nevada and Fort Smith Railroad and for the Missouri, Kansas and Texas Railway, and was also engaged in private practice.

In August, 1894, Mr. Hinckley was appointed Chief Engineer of the Choctaw, Oklahoma and Gulf Railroad (now the Rock Island Company), and constructed the line from McAllister to Oklahoma City. In January, 1896, he was engaged as Chief Engineer of Construction of the St. Louis and San Francisco Railroad, and had charge of the construction of many miles of this road, mostly in Oklahoma and Texas, including the lines from Bolivar to Osceola, Mo.; from Sapulpa to Oklahoma City, Okla.; from Sapulpa, Okla., to Sherman, Tex.; from Tulsa to Avard, Okla.; and from Sherman to Carrolton, Tex. On the Fort Worth and Rio Grande Railway, he was in charge of the construction from Brownwood to Brady, Tex., and supervised the contruction of the line from Hope to Ardmore, Mo., and from Oklahoma City to Quanah, Tex. He also supervised the construction of the Blackwell, Enid and Southwestern Railway, from Okeen, Okla., to Vernon, Tex.

In January, 1904, Mr. Hinckley was appointed Chief Engineer of the Frisco, remaining in this position until February, 1909. During this time he supervised the construction of the St. Louis, Brownville and Mexico Railway, the Colorado Southern, the New Orleans and Pacific, and the New Orleans Terminal Railway. During his service with this road (1896-1909), about 1 800 miles of the present "Frisco System" were built under his supervision. On his resignation as Chief Engineer, in February, 1909, he engaged in private practice, opening an office in St. Louis, Mo. His death, which occurred on February 20th, 1911, after a short illness, was caused by pneumonia.

Mr. Hinckley was closely identified with the development of what is now the State of Oklahoma through his work of railroad construction. He was a courteous man, kind and considerate to his subordinates and ever willing to promote the younger men in his employ; consequently, he had the esteem and confidence of his assistants to a marked degree. He was as studious as permitted by his active life and owned one of the best libraries in St. Louis. Although he did not seek general society, he was hospitable and friendly, especially to those of his Profession, and he will be greatly missed by all who knew him.

Mr. Hinckley was elected a Member of the American Society of Civil Engineers on May 6th, 1885. He was also a Member of the American Railway Engineering and Maintenance of Way Association, the American Society for Testing Materials, the Engineers' Club of St. Louis, and other societies.

LOUIS EDWIN HAWES, Assoc. M. Am. Soc. C. E.*

Died January 29th, 1911.

Louis Edwin Hawes, son of Henry E. and Frances E. (Wesson) Hawes, was born in Springfield, Mass., on January 27th, 1860, his mother being a sister of the late Daniel B. Wesson, head of the world-famous firm manufacturing the Smith and Wesson revolver.

His youth was passed in his native city, where, at the age of fifteen, as the eldest child of a widowed mother with six children, the stern duty of caring for and maintaining the family was suddenly thrust upon him. That this duty was faithfully performed is attested by the filial affection and deep family devotion which characterized Mr. Hawes throughout his life.

He studied at the Worcester, Mass., Polytechnic Institute, where he entered with zest into the sports of the day, being especially skilled in base-ball. He played the violin in the orchestra of the Institute. On this graduation, in 1882, with the degree of B. S., he at once began active work in his chosen profession. In the summer of 1882 he was Levelman and Assistant Engineer on the location survey for the Meriden and Cromwell Railroad in Connecticut, and later in the year, Assistant Engineer on water-works construction at Northboro, Mass. During 1883 and 1884, he was Assistant Engineer on the Wakefield, Stoneham, and North Attleboro, Mass., water-works. In 1885 he was Resident Engineer on the Norwood, Mass., water-works, and in 1886 served in the same capacity on the Juniper Hill Reservoir, at Rockland, Me., and was in charge of the surveys for a sewerage system at Hyde Park, Mass.

In 1887 he was employed as Resident Engineer on the construction of the Ayer, Mass., water-works, and, in 1888, as Assistant Engineer on the Dover, N. H., water-works, his work for the preceding five years having been under Mr. Percy M. Blake.

In 1889, Mr. Hawes began business for himself, opening an office in Boston, Mass. He continued in active practice as a civil and hydraulic engineer and as a contractor until his death, a period of twenty-two years. During this time he investigated, designed, and reported on or constructed new water-works systems or improved old ones in about forty towns and cities in New England and the West, and was frequently called as an expert in the appraisal of water-supply plants, in legal controversies, and in other matters involving hydraulic questions. Among the places where he did work may be mentioned, Avon, Needham, Provincetown, Lexington, Holden, Housatonic, Falmouth, Amesbury, Milford, Whitman, Edgartown, and Marion, in Massachusetts;

* Memoir prepared by George M. Warren, Esq.

Middlebury, Vt.; Alton, Ill., and Independence and Newton, Kans. He designed and reported on a system of sewerage and sewage disposal for Wakefield, Mass., and, in 1895, was Chairman of the Sewerage Commission of that town. At the time of his death he was Treasurer and Manager of the Edgartown Water Company.

Mr. Hawes was a man of pleasing personality, unswerving integrity, strong religious conviction, and, as an engineer, thoroughly reliable, painstaking, and observant of every detail. During a close professional association of more than ten years, the writer never heard him utter a violent or unchaste remark or speak ill of any one. One of his contemporaries writes:

"Mr. Hawes was a careful and painstaking engineer of ability and good judgment. Although modest and retiring in his manner, he gave to all matters entrusted to his attention a full consideration and the benefit of a well-balanced judgment. Whatever he did, he did well."

The Chairman of the Water Commissioners of Holden, Mass., referring to the monthly estimates prepared by Mr. Hawes, wrote: "They are models of clearness and mathematical accuracy."

On June 16th, 1886, Mr. Hawes married Miss Hattie M. Emerson, of Wakefield, Mass., who, with a daughter and a son, survives him.

Mr. Hawes was admitted to the New England Water-Works Association, on December 12th, 1888; and to the Boston Society of Civil Engineers, on June 20th, 1894.

He was elected an Associate Member of the American Society of Civil Engineers, on September 2d, 1896.